Anderson Dam Seismic Retrofit Project
Cofferdam Technical Memorandum

January 14, 2022

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SANTA CLARA VALLEY WATER DISTRICT
Anderson Dam Seismic Retrofit Project
SCVWD Project No. 91864005

COFFERDAM
Final Technical Memorandum

VERSION #4

Prepared By:
ADSRP DESIGN TEAM

URS

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Final Technical Memorandum

01/14/22, VERSION #4

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ACRONYM LIST

ADSRP	Anderson Dam Seismic Retrofit Project
ADTP	Anderson Dam Tunnel Project
AF	acre-feet
cfs	cubic feet per second
El.	elevation (1988 NGVD)
ft	feet
GDR	Geotechnical Data Report
gpm	gallons per minute
HLOW	high-level outlet works
LLOT	low-level outlet tunnel
LLOW	low-level outlet works
pcf	pounds per cubic foot
SCVWD	Santa Clara Valley Water District
TM	Technical Memorandum
USACE	United States Army Corps of Engineers

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1 INTRODUCTION

1.1 BACKGROUND AND PURPOSE

The Anderson Dam Seismic Retrofit Project (ADSRP) is being undertaken by the Santa Clara Valley Water District (District, SCVWD) to correct Anderson Dam deficiencies identified in previous studies. Construction of proposed improvements will require a cofferdam to intercept and bypass reservoir inflows around the work site through a diversion system so that excavation for, and construction of a new replacement dam, can proceed in the dry. Temporary pumping capability will be required to bypass reservoir inflows around the work site during the first year of embankment excavation (ADSRP Year 2) while the cofferdam, diversion system, diversion intake, and extension pipe are being constructed. During the last year of embankment construction (ADSRP Year 6), the temporary pumps will convey flows up to the high-level outlet works (HLOW) while the low-level outlet works (LLOW) is completed.

The purpose of this technical memorandum (TM) is to present the basis of design for the preferred cofferdam alternative including type, size and location of the cofferdam, the criteria for configuration and the capacity of the temporary reservoir bypass pumping system, and the criteria and methodology used for sizing of the diversion extension pipe. Previous versions (Version 0 and Version 1) of this TM presented the evaluation of potential cofferdam alternatives and developed the basis of design for a preferred cofferdam alternative. That alternative involved dredging the lakebed sediments in the wet to expose a stable foundation followed by backfilling of the excavation with granular backfill and installing a sheet pile wall through the fill to provide a seepage cutoff to rock. The dredging alternative was selected in previous versions (Version 0 and Version 1) of this TM because it offered a lower construction risk and had less construction uncertainty than the next alternative, which was a cofferdam constructed by displacing the lake bed sediment.

During the course of the design phase, additional dam safety deficiencies were identified including embankment transition zone quality, fault rupture across the embankment, and potentially liquefiable soils in the upstream dam embankment. The project was modified to address these deficiencies through removal of most of the existing Anderson Dam and replacement with a well-compacted, zoned embankment dam. Construction sequencing of the project no longer allows for the duration of time required to construct the previously preferred cofferdam but can accommodate a cofferdam built by displacing lakebed sediment. The current preferred alternative is a displacement cofferdam that would have a wider crest than described in the previous Cofferdam TMs to reduce construction risk and uncertainty described in the previous TMs.

This TM builds upon the revised design modifications; the previous Cofferdam TMs (URS, 2014, 2015, 2018, 2021a), and the Diversion Basis of Design (BOD) TM (URS, 2022) that presents the two temporary diversion systems, that will be needed during construction. The two diversion systems are the Stage 1 Diversion System constructed during the Anderson Dam Tunnel Project (ADTP) and the Stage 2 Diversion System constructed during ADSRP Year 2.

1.2 SCOPE OF WORK

The scope of work for this Cofferdam TM is to address the following:

1. Define the basis of design for the cofferdam, the bypass pumping requirements, and sizing of the diversion extension pipe that will extend from the Stage 2 Diversion System intake structure upstream past the cofferdam.

2. Make a recommendation whether the cofferdam should be designed as part of the Contract Bid Documents or whether the cofferdam can be designed by the Contractor as part of their temporary site works to a set of criteria provided in the Contract Specifications.

3. Assess design and construction risks associated with the proposed cofferdam.

4. Develop the design of the cofferdam for constructability evaluation and for scheduling and cost-estimating purposes. This includes descriptions of methodologies used, input parameters and assumptions made, and results of any analyses. It will also consider potential material sources and project schedule.

This TM does not address environmental or permitting issues associated with dewatering the reservoir nor does it address downstream handling, treatment (if needed), and release of water being pumped around the work site through the temporary bypass pumping system or passing through the Stage 1 or Stage 2 diversion systems.

1.3 ORGANIZATION OF TECHNICAL MEMORANDUM

The TM is organized as follows:

1. Section 1 presents an introduction and the scope of work.

2. Section 2 presents the basis of design

3. Section 3 presents the rationale for designing the temporary cofferdam as part of the project design rather than making it part of the Contractor's temporary works.

4. Section 4 presents the design for the preferred cofferdam.

5. Section 5 presents limitations of the TM.

6. Section 6 lists references reviewed in preparation of this TM.

2   BASIS OF DESIGN

2.1   PURPOSE

The primary purpose of the cofferdam is to intercept reservoir inflows in order to allow a dry work area immediately upstream of the dam during the dry season. Storage created behind the cofferdam dam will initially serve as a forebay for a temporary system of pumps that will convey bypass flows to the Stage 1 Diversion System[1] while the Stage 2 Diversion System is being constructed (URS, 2022). The Stage 2 Diversion System (URS, 2022) from upstream to downstream includes:

- About 780 feet of 10-foot diameter extension pipe,
- Diversion Intake Structure,
- Upstream end of the low-level outlet tunnel (LLOT) - 160 feet of excavated 18-foot horseshoe tunnel finished with a concrete encased 12-foot diameter steel followed by 115 feet of excavated 24-foot horseshoe tunnel finished with a 19-foot horseshoe reinforced concrete liner tying into the Stage 1 Diversion System at Station 3+80, and
- The Stage 1 Diversion System downstream of Station 3+80.

The 10-foot diameter extension pipe will be installed to act as a passive spillway conveying water from the cofferdam forebay to the Stage 2 Diversion System. The temporary extension pipe will be used to convey flows during the fall of the first embankment construction season (ADSRP Year 2), the spring and fall of the second, third, and fourth embankment construction seasons (ADSRP Years 3, 4, and 5), and the spring of the fifth embankment construction season (ADSRP Year 6). The temporary bypass pump system will be required again during fifth embankment construction (last summer of cofferdam operation) as the 10-foot diameter extension pipe and diversion intake structure are removed and the new sloping intake is connected to the LLOT.

The reservoir will be initially lowered to the invert (El. 450[2])of the Stage 1 Diversion System by May 1 or earlier of the first embankment construction season (ADSRP Year 2), depending on the precipitation that season. The Contractor will then be responsible to construct the cofferdam and design, install, maintain and operate a bypass pumping system with a specified capacity until completion of the Stage 2 Diversion System.

2.2   DIVERSION CONCEPT

The configuration and operation for the cofferdam, bypass pumping system, and diversion extension pipe are discussed below. Design of the cofferdam is discussed in Section 4, with supporting geotechnical data in Appendices A and B.

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[1]The Stage 1 Diversion System is the low-level diversion system that is being constructed as part of the ADTP. The Stage 1 Diversion System includes an upstream trash rack, 330-foot 8-foot diameter lake tap pipe, 1,050 feet of finished 19-foot horseshoe tunnel lined with reinforced concrete, 336-feet of 13-foot diameter steel pipe lined tunnel, 38 feet of 13-foot steel pipe that bifurcates into two 11-foot steel pipes that terminate in 11-foot fixed cone valves in a Diversion Outlet Structure (URS, 2022).

[2] All elevations in this TM reference the North American Vertical Datum of 1988 (NAVD88).

1. A small forebay, in combination with a bypass pumping system, will be used to provide protection for the upstream work area during the dry season of the first year of embankment excavation (ADSRP Year 2), which will also be the year the cofferdam is constructed. The forebay will operate between El. 460.0 and El. 467.0 (invert elevation of sheet pile notch; see Section 4.5), which equates to an operating storage of approximately 400 acre-feet (AF) (see Figure 4-1 for plan and Figure 4-2 for profile along cofferdam axis). The total storage behind the cofferdam is about 500 AF.

2. A bypass pumping system consisting of two (15 cfs) diesel-powered trash pumps – one as operating unit and one as a backup. Both could be used if available, but both should not be relied upon to be available. A minimum set of performance requirements will be specified for the pumping system. If the minimum requirement was two pumps of equal size (one as a backup), each with a capacity of 15 cfs (with cofferdam level at El 467 and the discharge point at El 450), then DV350c (or similar) trash pumps rented from Rain-for-Rent (or other supplier) would satisfy the requirements. With between 15 cfs (single pump operating) and 30 cfs (both pumps operating) pumping capacity and 400 AF of storage, there is a low risk of overtopping the cofferdam between May and November (based on the data provided in Section 2.3). Operation of the bypass pumping system would end when the Stage 2 diversion system is completed, and reservoir inflows are passing into the 10-foot diversion extension pipe. A memorandum describing the basis of selection of this pump is included as Appendix C. The memorandum in Appendix C covers a wider range of pumping capacities than indicated here.

   The bypass pumping system will convey water from the pumps at the cofferdam forebay through approximately 950 feet of 30-inch diameter pipe to a discharge point at the upstream end of the Stage 1 Diversion System (Figure 4-1). The pumps will discharge into a manifold system that will connect to the 30-inch pipe. During the fifth year of embankment construction (ADSRP Year 6) the pumps will lift the water to the upstream intake of the high-level outlet works (HLOW) at El. 528.

3. At the start of winter (date to be determined based on weather forecasting[3]), the 10-foot gate at the Diversion Intake Structure would be partially shut to allow inflows to fill the cofferdam forebay and spill over the riprap lined cofferdam spillway while at the same time releasing a minimum of 5 cfs into Coyote Creek through the Stage 2 Diversion System. Releases over the cofferdam spillway would continue until the approximately 100 AF of storage between the cofferdam and the interim-stage dams has filled and inflows begin to pass through the top of the Diversion Intake Structure. With the water surfaces equalized, the 10-foot gate would be fully closed and all winter flood flows would be released through the diversion system. The length of time required to equalize the water levels will depend on inflows into the reservoir area.

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[3] Since the purpose of the 10-foot extension pipe is to allow earthwork to extend into the fall shoulder season (October 15 to December 31), the determination of when to shutoff flow through the 10-foot extension pipe would be dependent on the occurrence of a storm large enough to produce reservoir inflows that exceed the extension pipe's 950 cfs flow capacity. The intent would be to shut off the 10-foot extension pipe for the winter when weather forecasts predict a storm large enough to potentially produce such inflows is approaching the project area.

4. At the start of the subsequent construction season, when weather forecasting indicates that the storm season is over and the potential for storms that could produce reservoir inflows greater than 950 cfs is unlikely, the 10-foot gate at the Diversion Intake Structure would be opened to allow inflows to pass through the 10-foot extension pipe into the Stage 2 Diversion System. The water downstream of the cofferdam (100 AF) would be released through the Phase 2 Diversion System via a 36-inch gated opening in the side of Diversion Intake Structure that would allow water to be drawn down to El. 453.

5. The cofferdam must be maintained until all upstream construction is complete and until access is no longer needed to stockpile areas in the reservoir area (SA-K, SA-L, SA-H, SA-C, and SA-D, see URS 2021b), the Reservoir Disposal Area, and the Packwood Gravel Borrow Pit at the end of the fifth embankment construction season (ADSRP Year 6). In the event that an additional embankment season is needed, flood flows could be routed through the HLOW intake structure that will act as the diversion system during the sixth year of embankment construction (ADSRP Year 7).

2.3   OVERTOPPING RISK

During construction, inflows to the reservoir behind the cofferdam will consist of releases from Coyote Reservoir and inflows from the watershed between Coyote and Anderson dams. The risk of overtopping of the cofferdam during spring and fall shoulder seasons[4] was estimated using the following assumptions and data:

*   Releases from Coyote Reservoir were assumed to be 5 cfs.
*   Inflow from the watershed between the two dams was estimated from the gaged flow upstream of Coyote Reservoir assuming equal flow per unit area.
*   The flow was obtained from the USGS website for gage "USGS 11169800 COYOTE C NR GILROY CA". Daily streamflow data were available from October 1, 1960 through September 29, 1982 and from October 1, 2004 to December 31, 2018. Flow was daily average flow and annual maximum instantaneous flow. 15-minute stream flow data were available from January 1, 2005 to December 31, 2018.

Flow data were analyzed for the months of April through December. The 15-minute data were used to develop ratios between the peak 15-minute flow and the peak daily flow for several runoff producing events in the record. One storm (December 23, 2012) with a ratio (4.1) in the mid to upper range of the ratios was selected to synthesize hydrographs for all the runoff producing events from April to December found in the daily flow record.

A hydraulic model was created using the U.S. Army Corps of Engineering Hydraulic Engineering Center – River Analysis System (HEC-RAS) Version 5.0.5. The model was used to simulate the routing of the synthetic rainfall events through Coyote and Anderson Reservoirs and through 6-foot, 8-foot, and 10-foot diameter diversion extension pipe sizes. Events resulting in a peak daily flow of less than 220-cfs at the Coyote Creek near Gilroy gage (USGS gage 11169800) would be

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[4] The spring shoulder season is defined as April 1 to May 15 and the fall shoulder season is defined as October 15 to December 31

passable by a 6-foot diameter diversion extension pipe and were thus not considered in the hydraulic modeling. For comparison, a 30 cfs bypass pump system can pass an event with daily peak flow of about 110 cfs at USGS gage 11169800 given the storage assumptions for Coyote Reservoir and the cofferdam forebay (see Appendix D for details of the storage assumptions).

For the streamflow events that were considered in the modeling, Table 2-1 lists the total number of storms during the first and last halves of each month, the corresponding number of storms that are bypassed for each pipe size, and the number of storms that would overtop the cofferdam sheet pile (El. 467).

Table 2-1   Hydraulic Routing Results Summary for Storms Greater Than 220 cfs from April Through December

Description	Apr 1-15	Apr 16-30	Oct 1-15	Oct 16-31	Nov 1-15	Nov 16-30	Dec 1-15	Dec 16-31	Total
Number of Storms	8	3	2	0	2	1	6	11	33
Bypass with 6' Pipe	0	0	1	0	0	0	1	4	6
Bypass with 8' Pipe	2	2	1	0	1	1	1	5	13
Bypass with 10' Pipe	3	2	1	0	1	1	3	5	16
Cofferdam overtopped	5	1	1	0	1	0	3	6	17

Note: There were no storms greater than 220 cfs in the months of May, June, July, August, or September.

As shown in Table 2-1, it is estimated that 6-foot, 8-foot, and 10-foot diversion extension pipes would bypass:

- April 1 to December 31 - 18%, 39%, and 48% of historic storms greater than 220 cfs, respectively
- April 15 to December 15 - 14%, 43%, and 57% of historic storms greater than 220 cfs, respectively
- April 15 to November 30 - 13%, 62%, and 62% of historic storms greater than 220 cfs, respectively

The number of historic storms that would pass through the diversion extension pipes that would not have been passed by the 30 cfs bypass pumping system was estimated by counting up the number events recorded at USGS gage 11169800 that had peak daily flows greater than 110 cfs and less than 220 cfs. The resulting number of storms was 14 with 8 occurring in April, 2 in November, and 4 in December. Table 2-2 lists the storms greater than 110 cfs that would be bypassed or result in overtopping to capture the estimated benefit of the diversion extension pipe over the 30 cfs bypass pump system.

Table 2-2   Hydraulic Routing Results Summary for Storms Greater Than 110 cfs from April Through December

Description	Apr 1-15	Apr 16-30	Oct 1-15	Oct 16-31	Nov 1-15	Nov 16-30	Dec 1-15	Dec 16-31	Total
Number of Storms	15	4	2	0	2	3	7	14	47
Bypass with 6′ Pipe	7	1	1	0	0	2	2	7	20
Bypass with 8′ Pipe	9	3	1	0	1	3	2	8	27
Bypass with 10′ Pipe	10	3	1	0	1	3	4	8	30
Cofferdam overtopped	5	1	1	0	1	0	3	6	17

Note: There were no storms greater than 220 cfs in the months of May, June, July, August, or September.

As shown in Table 2-2, it is estimated that 6-foot, 8-foot, and 10-foot diversion extension pipes would bypass:

- April 1 to December 31 - 43%, 57%, and 64% of historic storms greater than 110 cfs, respectively
- April 16 to December 15 - 33%, 56%, and 67% of historic storms greater than 110 cfs, respectively
- April 16 to November 30 - 36%, 73%, and 73% of historic storms greater than 110 cfs, respectively

More detail on the analyses is included in Appendix D.

The modeling results demonstrate that a diversion extension pipe substantially reduces the potential for overtopping of the cofferdam in April, November, and December and thereby increases the number of days that would be available during the shoulder seasons for work in the reservoir area. Based on available historic flow data, a 10-foot diversion extension pipe will bypass 30 of 47 events that would not be able to be bypassed using a 30 cfs pumping system. It is recommended that moving forward into 90% design that a 10-foot diameter diversion extension pipe be incorporated into the project to maximize the reduction of risk of cofferdam overtopping.

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3 CONTRACTOR OR OWNER DESIGN

3.1 COFFERDAM CLASSIFICATION

USACE EM 1110-2-2300 (USACE, 2004) defines a major cofferdam as that which upon failure would cause major damage downstream and/or considerable damage to the permanent work. Minor cofferdams are defined as those which upon failure would result in only minor flooding of the construction work.

The direct cost of constructing the cofferdam and associated dewatering can be a significant portion of the final project's cost. In addition, the vulnerability of the work downstream of the cofferdam and the impact of failure of the cofferdam on the construction schedule are significant. Therefore, considering the schedule impact and associated direct and indirect costs of a failure of cofferdam for the ADSRP, the cofferdam can be classified as a major structure.

3.2 OWNER DESIGN

Design of a cofferdam by the contractor is normally allowed only when the construction schedule provides ample review and design time to ensure a competent and safe design or where no major damage or significant delays to the project would occur from a failure of the cofferdam. EM 1110-2-2300 and ER 1110-2-8152 (USACE, 1994) recommends that major cofferdams be planned, designed, approved and constructed to the same level of engineering competency as for main dams, which for the ADSRP would mean they are designed as part of the Contract Documents for the project. Owner design also provides for uniformity in bidding, since the contract plans and specifications would then include the major features of the cofferdam.

Design considerations will include minimum required top elevation, hydrologic records, hydrographic and topographic information, subsurface exploration, seepage control, stability and settlement analyses, maintenance of freeboard, and sources of construction materials. Placement and compaction procedures as well as other pertinent construction aspects, including construction and monitoring requirements, are to be covered in the contract plans and specifications. Performance specifications for contractor-furnished dewatering and bypass systems will be developed based on the project's design criteria.

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4 COFFERDAM DESIGN

4.1 GEOTECHNICAL CONDITIONS

4.1.1 Field Exploration Program

The field exploration program for the cofferdam consisted of four borings (designated CD-1 through CD-4), drilled (from a barge mounted drill rig) along the proposed alignment of the cofferdam shown in Figure 4-1. A fifth boring (CD-2a) was drilled adjacent to the location of boring CD-2 to collect undisturbed samples of the lakebed sediments. The purpose of the borings was to:

1. Define the subsurface conditions beneath the alignment of the cofferdam, particularly the depth of accumulated lake bottom sediments and the thickness of underlying alluvium/colluvium, and

2. Evaluate the geotechnical characteristics of the sediments and alluvium/colluvium and obtain undisturbed samples for laboratory testing.

An idealized subsurface profile along the proposed alignment of the cofferdam is shown in Figure 4-2. This section indicates the locations of the four borings and the estimated subsurface profile. The reservoir level at the time of drilling, and the proposed top of the cofferdam (El. 465.0) are also shown on the profile for reference. The profile was constructed by interpolation between the borings and comparison with the 2014 bathymetry. In general, subsurface conditions across the floor of the reservoir consist of up to 38 feet of soft, plastic normally-consolidated lakebed sediments overlying 3 to 5 feet of alluvium (Qal) or colluvium (Qc), which in turn overlies shale and serpentinite (Franciscan Formation) bedrock. Soft soils taper towards both abutments as bedrock topography rises out to the old river channel. In boring CD-4; however, 8 feet of colluvium was recorded, likely in a localized area. Comparing the 1988 and 2014 lakebed topographic/bathymetric surveys indicates minimal sedimentation has occurred in the reservoir at the proposed cofferdam location over the 26 years between the surveys.

Table 4-1 provides a summary of the conditions encountered at the borings. A description of the drilling methods and logs of the holes are presented in the Geotechnical Data Report (GDR) (URS, 2021c). It is noteworthy that a 4-inch casing penetrated 28 feet and 24 feet into the soft lakebed sediments in Borings CD-2 and CD-3, respectively, solely under the self-weight of the casing string.

Table 4-1. Boring Summary

BORING	MUDLINE ELEVATION (FT.)	TOTAL DEPTH (FT.)	TOTAL SEDIMENT THICKNESS (FT.)	TOTAL Qc/Qal THICKNESS (FT.)	TOTAL DEPTH INTO ROCK (FT.)
CD-1	458.2	51.0	5.0	Qal/Qc: 2.5	43.5
CD-2	451.9	65.0	35.5	Qal: 4.5	25.0
CD-3	451.3	65.0	34.5	Qal: 2.5	28.0
CD-4	476.3	50.0	9.0	Qc: 8.0	33.0

Six approximately evenly spaced 2-foot-long thin-wall tube samples were obtained from the soft sediments in Boring CD-2a. Samples were retrieved using a push-piston sampler, which has a piston that is initially locked into place at the end of the thin wall tube to prevent sediment entering. After the sampler was pushed to the sampling depth, hydraulic pressure in the drilling fluid in the rods was increased to release a trigger that allows the thin wall tube to advance 2 feet. The sampler was then retrieved, a new tube installed, and the piston reset. The sampler was then pushed to the next sampling depth and the procedure repeated.

4.1.2 Laboratory Testing

Laboratory testing consisted of unconsolidated-undrained (UU) and isotropically consolidated undrained (ICU) triaxial strength tests, consolidation tests, moisture content and index property (Atterberg limits) measurements. The results of these tests are presented in the GDR (URS, 2021c). The sediments extruded from the six thin-walled-tubes in the laboratory had an appearance similar to that of a "young" San Francisco Bay Mud.

The sediments have an in-place wet density of about 101 pcf. The moisture content ranged from 42 to 68 percent. Atterberg limits were measured in each of the six samples. The measured ranges of liquid and plastic limits were 52 to 65 and 27 to 32 respectively, giving a plasticity index range of 23 to 33. All of the Atterberg limits test results plot along the A-line with liquid limits greater than 50. The sediment therefore classifies as a CH, MH or CH-MH, silty clay/clayey silt of high plasticity. The Atterberg limits test report summary is included in Appendix B.

The sediments short-term post-construction strength was characterized by an undrained shear strength ratio of 0.23 that was estimated from the ICU strength tests and compared with the UU strength tests. It is noteworthy that the UU sample for the top tube (No. 1 at 3-4 feet) was so soft that it slumped under its own weight. The UU sample for the second tube (No. 2 at 7-8 feet) was clearly disturbed during sampling and was also untestable.

The ICU and UU test results are shown graphically on Figure A-1 (Appendix A). Summary sheets of the ICU and UU test results are included in Appendix B, with the complete test results presented in the GDR (URS, 2019). The ICU samples were consolidated to stresses greater than the insitu condition so that the measured strength is for a normally-consolidated condition, unaffected by sample disturbance. The ratio of undrained shear strength (s_u) to isotropic consolidation stress (p') is about 0.34.

In selecting the appropriate field mobilized strength for use in the stability analyses, several adjustments must be made to the laboratory strength value described above.

1. Anisotropic consolidation stresses: Consolidation under anisotropic conditions (instead of the isotropic consolidation conditions used in the laboratory) can result in slightly lower strengths, with reductions on the order of 5-10% (Degroot and Ladd, 2012).

2. Sensitivity and disturbance: Normally-consolidated clayey soils can have internal structure originating from their depositional environment, the destruction of which can result in loss of undrained strength during shearing. The loss of strength due to remolding or to shearing to a fully residual condition can be significant. However, given that this structure will be

built incrementally, with the displacement of the sediment occurring in a series of relatively small bearing capacity failures along the advancing front of the fill, mobilized strengths will vary between peak, remolded, and residual conditions in different areas at different times.

3. Rate of strain: Laboratory strength testing typically happens at greater strain rates than field stability failures, with time to failure measured in hours in the laboratory compared to days or weeks in the field. The higher rate of strain can result in laboratory shear strength values up to 10% greater than field values. Note that the planned construction sequence for this structure relies on frequent bearing capacity failures so time to failure here may be closer to laboratory conditions than for a typical embankment on a soft foundation.

Considering the potential range of reductions, we selected a total reduction of about 33%, resulting in a field mobilized undrained strength ratio of 0.23. As discussed below, the use of this value results in a worst-case design condition where the strength is not quite low enough to allow the fill to fully penetrate through the existing reservoir sediment, resulting in a design section that leaves a potential weak layer in place beneath the structure.

Table 4-2. Sediment Geotechnical Properties

Saturated unit weight (pcf)	100
Natural moisture content (percent)	42-68
Silt/clay content (percent)	60/40
Liquid limit (percent)	52-65
Plasticity index (percent)	23-33
Liquidity index	0.5-1.7
Overconsolidation ratio	1.0
Compression index C_c	0.6
Effective stress friction angle ICU	27°
Laboratory undrained shear strength ratio	0.34
Field mobilized undrained shear strength ratio	0.23[1]

Notes:
1. For end-of-construction stability analysis the undrained strength is approximated as elevation-dependent, using the buoyant unit weight and the selected undrained strength ratio.

4.2 COFFERDAM DESCRIPTION

The cofferdam will be located approximately 1,250 feet upstream of the replacement dam axis and approximately 500 feet upstream of the replacement dam excavation as shown on Figure 4-1. Construction of the cofferdam embankment across the lakebed would be accomplished immediately following reservoir lowering by end-dumping materials excavated during the ADTP that were stockpiled in the boat ramp parking lot (Stockpile Area B), displacing the soft lakebed

sediment. Filling would be arranged to displace a large volume, if not most, of the soft lakebed sediment within the cofferdam footprint. The objective of maximizing displacement of the sediment is to provide for post-construction stability of the embankment and minimize settlement of the cofferdam crest. The sheet pile cutoff would be driven into rock after completion of the fill placement.

The displacement method of construction has been used since the early 20th Century for building road and rail embankments, levees, and dikes on soft soils. A variety of design manuals (Pihlainen 1963, MacFarlane 1969, USACE 1977, USACE 1987, Holtz 1989) describe methods of construction where earth embankment structures are advanced across soft ground (including young sediments and peat) by deliberately overloading the leading edge of a fill in order to induce a bearing capacity failure that displaces soil from beneath the fill. The most significant case histories are the construction of the San Francisco Bayshore Freeway (Highway 101) over 60 feet of soft Young Bay Mud in 1955 (Smith, 1955) and the construction of the Great Salt Lake Causeway over 120 feet of soft clay in 1959 (Lambrechts and Kinner 1988). A more recent example of use of the displacement method was placement of excess excavated materials in a disposal area over up to 15 feet of soft lake sediment at a dam replacement project in the San Francisco Bay area. While filling was progressing in the disposal area, mud waves and scarps associated with bearing capacity failures periodically appeared and were buried as filling operations progressed. Excerpts from some of the references are included in Appendix E.

The main risk inherent to the displacement method is that the extent and uniformity of sediment displacement are difficult to control. Incomplete or non-uniform displacement of the sediment can result in irregular or excessive settlement of the crest. However, in the absence of highly irregular conditions and in view of the temporary nature of the structure, settlement of the crest could be managed through periodic maintenance during construction.

Construction factors influencing the degree and uniformity of sediment displacement include the shape of the advancing face of the fill, the rate of fill placement, and work stoppages and interruptions. Displacement of the sediment may be enhanced by periodic dredging of mud waves to the side and in front of the advancing fill, and by overbuilding the height of the fill.

Construction would be accomplished by end-dumping fill directly onto the sediment starting at the left (southeast) end of the cofferdam alignment and continuously advancing the fill to the right end. To achieve maximum penetration of the fill and relatively uniform displacement of the sediment, the fill should be advanced using a wedge-shaped front as illustrated on Figure 4-1. This approach is intended to cause the displaced sediments to heave laterally away from the fill advancing face rather than build up in front of the advancing face. It may be necessary to periodically remove excessive heave (mud waves) from in front of the advancing face using a dragline excavator or other suitable equipment to avoid major entrapment of soft sediment within the cofferdam fill. However, removal of mud waves might cause localized instability of the embankment. The shoulders of the cofferdam embankment could also be overbuilt and then cut back to flatter slopes to enhance post-construction stability. Mud waves upstream of the advancing fill or beyond the downstream limits of the fill may help with stability so those should not be removed if possible.

Post construction borings should be drilled to assess the depth of sediment displacement achieved. Records of backfill quantities placed would also be used to assess the approximate configuration of the embankment below the surface.

The data used for the overtopping risk described in Section 2.3 also has application to construction of the cofferdam. Based on a review of that data and assuming inflows into the reservoir need to be less than 30 cfs to build the cofferdam, cofferdam construction could start April 15th in 23 of the 35 years. Cofferdam construction would be delayed until May 1st in 9 of the 35 years and until May 15th in 3 of the 35 years.

Construction of the 10-foot bypass pipeline following cofferdam construction will require excavating a small portion of the right abutment and right side of the cofferdam fill. The alignment and profile of the bypass pipe are based on 1949 pre-construction topography so that the pipe foundation would be either colluvium or bedrock. The concrete encasement at the right end of the cofferdam that would be founded on highly weathered Franciscan Formation will need to be tied into the sheet piles to provide a continuous seepage barrier at the connection.

4.3   STABILITY AND SEISMIC DEFORMATION ANALYSES

Stability analyses were evaluated for two situations; the first being that of the cofferdam, and the second being that of the sediment downstream of the cofferdam through which the dam excavation will be made.

4.3.1   Displacement Cofferdam

Limit equilibrium slope stability analyses were performed to assess the factor of safety for the displacement cofferdam. The analyzed cross section is shown in Figure 4-3. The section is representative of the midpoint of the cofferdam alignment where the depth of lakebed sediment is the greatest (38 feet). Three scenarios were considered for the slope stability analyses: end of construction, long term, and rapid flood loading. The dumped backfill was assigned a unit weight of 120 pcf and an effective friction angle of 30 degrees based on past project experience. Properties from the Embankment Basis of Design TM were used for the alluvium and bedrock (URS, 2020d). The alluvium was assigned a unit weight of 120 pcf and an effective friction angle of 35. The bedrock was assigned a unit weight of 150 pcf and an effective friction angle of 38 degrees. The strength of the sheet piles was ignored in the stability calculations. However, the effect of the sheet pile to lower the phreatic surface is modeled. The cofferdam, being located approximately 500 feet upstream of the excavation for the dam foundation, is judged to be far enough away that the dam excavation will not lead to instability of the cofferdam.

The cofferdam could be used as an access route to the upstream right abutment, stockpile areas SA-C, SA-H, SA-L, and the Packwood Gravel Borrow Pit. Therefore, traffic loadings are also modeled in the slope stability analysis. The stability analysis uses a fully loaded Caterpillar 777 dump truck.

A similar displacement backfill project was performed to construct a 30-foot-wide fill section of the San Francisco Bayshore Freeway (Highway 101) over 60 feet of soft Young Bay Mud in 1955 (Smith, 1955). It was reported that construction of this fill section resulted in nearly 100 percent displacement of the soft young bay mud. As it was suggested that not all soft soil could be displaced by end dumping, a 3-foot-thick layer of soft sediment was assumed to be left in place beneath the

proposed excavation fill in the stability analysis. For the stability analysis, the width of the top of the constructed cofferdam was assumed to be 60 feet. The plans call for the crest to be built to a top width of 80 feet (Figure 4-3).

For the end of construction analysis, the factor of safety will be near unity by definition as the dumped fill will reach equilibrium. The result of the end of construction analysis is used to determine if the assumed depth and size of the cofferdam are reasonable. The results of the stability analyses are summarized in Table 4-3. Figure 4-4 shows that the calculated factor of safety for the end of construction condition is about 1.1, which is consistent with the method of construction resulting in a marginally stable condition. Note that selecting a lower undrained strength for the reservoir sediment would result in the fill fully penetrating the reservoir sediment (i.e., with an end-of-construction factor of safety less than 1.0 unless all sediment has been removed from beneath the fill).

Figure 4-5 shows that the factor of safety increases to about 1.5 or better as the materials consolidate and gain strength, and Figure 4-6 shows that rapid flood loading assumptions result in a factor of safety of about 1.3 for a short-term condition. Dissipation of excess pore pressures after fill placement may take days or weeks depending on permeability and thickness of soft sediment remaining in place beneath the fill. Analysis of rapid drawdown conditions shows a factor of safety of about 0.9 (see Figure 4-7) for an assumed instantaneous drawdown with no drainage allowed in the fill, indicating that repairs to the slopes may be required after such an event. There is no mechanism by which the water upstream of the cofferdam could be drawn down quickly (aside from a breach of the cofferdam when the temporary bypass pumping system is being used) so this is a highly unlikely scenario for the upstream slopes. The downstream slopes may experience drawdown loading when the diversion extension pipe is opened and the area downstream of the cofferdam is dewatered at the start of each construction season, so minor repairs to the downstream slope may be required as part of preparations to begin embankment work each season.

Simplified seismic deformation analyses were conducted using the 100-year construction earthquake event (PGA = 0.36 g). Pseudo-static stability results shown on Figure 4-8 show yield coefficients ranging from 0.07 to 0.09. Using the Bray and Maced (2019) method, movements ranging from 0.6 to 1.6 feet were calculated. Using the Makdisi and Seed (1978) method as modified by FERC, movements ranging from 0.3 to 1.6 feet were calculated. Based on these results, the cofferdam is expected to experience moderate distress during a 100-year earthquake event. Based on the geometry of the structure and the slip surfaces, the movements are likely to manifest mostly as widening of the embankment with minor associated settlements. Repairs would likely be required following this earthquake event.

Table 4-3. Summary of Stability Analyses for the Cofferdam

SCENARIO	FACTOR OF SAFETY
End of Construction	1.07
Long Term	1.53
Rapid Flood	1.28
Rapid Drawdown	0.86

4.3.2 Dam Excavation Slope Through Sediment

During the removal of Anderson Dam, excavation/dredging into the lake sediment will be required in order to provide enough working space to remove the potential liquefiable materials from the upstream shell foundation. Due to the soft characteristics of the lake sediment, a relatively flat slope will be required. Slope stability analyses were performed to determine the required slope to achieve an adequate factor of safety of about 1.3 for the temporary condition. The material properties of the lake sediment are presented in Table 4-2. The drained and undrained strengths are used in the long term and end of construction analyses, respectively. Table 4-5 summarizes the results of the stability analyses that accounts for different excavation slopes and different depths of natural drainage/dewatering of the slope. The analyses assuming no dewatering model the phreatic surface at the excavated ground surface, essentially assuming that the excavation is kept dry, but no drainage from the remaining sediment occurs. The range of dewatering depths allow for some natural drainage to occur as water in the sediment downstream of the cofferdam seeps into the excavation and is pumped out. Based on these results, the excavation slope should be flatter than 5H:1V, depending on the effectiveness of the dewatering. Excavation of the dam foundation below El. 465 feet is not anticipated to occur until late summer during second season of embankment excavation (ADSRP Year 3).

Table 4-5. Summary of Stability Analyses for the Excavated Slope in Lake Sediment

SLOPE	DRAINED STRENGTH ANALYSIS FS			UNDRAINED STRENGTH ANALYSIS FS
	No Dewatering	3-ft Dewatering	5-ft Dewatering	No Dewatering
5H:1V	0.93	1.13	1.24	0.89
8H:1V	1.54	1.80	1.94	1.22
10H:1V	1.87	2.21	2.38	1.43

4.4 MATERIAL SOURCES

The end-dumped backfill will be materials excavated during ADTP from the Diversion Outlet Portal, discharge channel, Coyote Creek modifications, Diversion Outlet Structure foundation, and tunnel and shaft excavation. The excavated materials consist of alluvium, colluvium and highly to completely weathered Franciscan Bedrock Formation and Santa Clara Formation. The maximum

size aggregate will also be limited to approximately 3 to 4 inches to reduce segregation and to produce a fill through which the sheet pile can be driven. Processing (by grizzly) may be needed to produce this material gradation.

The ADTP excavations will produce approximately 200,000 cubic yards (cy) of materials, which is about four times greater than the 45,000 cy estimated to be required to construct the cofferdam. Excess excavation materials will be used to build an access road to the Reservoir Disposal Area.

4.5 SEEPAGE CONTROL/CUTOFF WALL

Seepage under and through the cofferdam will need to be controlled. Two methods of providing seepage control are (1) placement of an impervious synthetic lining or a low permeability clay blanket covering the upstream surface of the cofferdam embankment or (2) installation of a vertical cutoff wall through the cofferdam fill.

The integrity and durability of an impervious or low permeability covering approach (1) will be difficult to achieve and verify. A vertical steel sheet pile cutoff (method 2) is a safer and more positive method of controlling seepage through the cofferdam fill and foundation and, on this basis, is the recommended method for seepage control.

The sheet pile wall will be located 10 feet downstream of the upstream edge of the crest to provide room on the crest for installation and removal of the sheet piles, and for access across the cofferdam during construction. The tops of the sheets will generally be set at El. 468 (3 feet above the crest of the cofferdam) to facilitate later removal and to provide additional freeboard. A 37-foot-wide notch (invert El. 467) located in the central portion of the sheet pile will act as a spillway in the event that inflows are sufficient to cause the water surface elevation in the forebay to exceed El. 467. The discharge capacity of the 37-foot wide, 1-foot-deep notch is estimated to be about 160 cfs[5]. The discharge capacity 1 foot (El. 469) and 2 feet (El. 470) above the top of the sheets is estimated to be about 1,560 cfs[6] and 3,960 cfs, respectively.

Vibratory hammers will likely be satisfactory to drive the sheet piles. If the tip elevation of the sheets is not achieved through the backfill, impact hammers may be used. Predrilling or jetting will not be allowed because these activities may disturb the fill.

Where the 10-foot bypass pipeline crosses the cofferdam alignment it will be encased in reinforced concrete, with the concrete extending to overlap with the sheet piles.

The sheet pile wall and cofferdam embankment materials, while effective at controlling seepage, will not be impervious and some leakage will likely occur, especially during periods inflows are being conveyed around the cofferdam through the bypass pumping system and reservoir level is higher than during periods when inflows are being conveyed through the diversion extension pipe.

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[5] The discharge capacity of the sheet pile spillway is based on the sharp-crested weir equation, a discharge coefficient of 3.0, and a weir length of 52.9. The weir length is based on the length along edge of 8 pairs of AZ36-700 sheet pile that are specified to be used. The length of a pair of AZ36-700 piles has length along the sheet pile edge of 77.29 inches compared to a width of 55.12 inches.

[6] The weir length of the sheets with top El. 468 is 370 feet and is based on the length along edge of 56 pairs of AZ36-700 sheet pile that are specified to be used.

The contractor may use sump pumps on the downstream side of the cofferdam to pump seepage back to the upstream side where it can drain through the bypass pipeline.

4.6   PROTECTION OF COFFERDAM AGAINST OVERTOPPING

In order to allow overtopping during the precipitation season without causing excessive erosion or washout of the cofferdam, the water level on the downstream side of the cofferdam will be equalized with the water level on the upstream side of the cofferdam at the end of the dry season by closing the 10-foot stop gate to shut off the diversion extension pipe and allowing water to pass over the sheet pile spillway from the upstream side of the cofferdam to the downstream side until the water level on both sides of the sheet pile is at El 467 feet at which point reservoir inflows would pass over the diversion intake structure crest into the Stage 2 Diversion System. The sheet pile spillway and riprap lined channel across the cofferdam crest is also intended to protect the cofferdam from erosion if overtopping were to occur when the area downstream of the cofferdam is dry.

4.7   MAINTENANCE DURING CONSTRUCTION

As described above, there is potential for minor ongoing settlement or raveling of the cofferdam slopes during construction operations. The contractor can place additional disposal fill or haul road surfacing material to maintain the elevation profile of the haul road, and additional disposal fill at the edges of the crest to maintain the width as needed. The 80-foot crest width allows space for minor slumps or raveling at the edges to occur without impacting the structure's use as haul road.

Repairs or resurfacing are also likely to be required at the start of each construction season after the reservoir is lowered and the area downstream of the cofferdam is dewatered.

4.8   END OF CONSTRUCTION

The current dead pool elevation of El. 488 will be maintained with the new sloping intake structure. As the new outlet works has no provision for making releases below El. 488, there is no reason to remove the cofferdam. Removal of the sheet pile cutoff for its salvage value can be a contractor option. Confirmation with the SCVWD's Operations Unit will be needed to obtain their input on sheet pile and cofferdam removal.

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5 LIMITATIONS

URS represents that our services were conducted in a manner consistent with the standard of care ordinarily applied as the state of practice in the profession within the limits prescribed by our client. No other warranties, either expressed or implied, are included or intended in this Technical Memorandum.
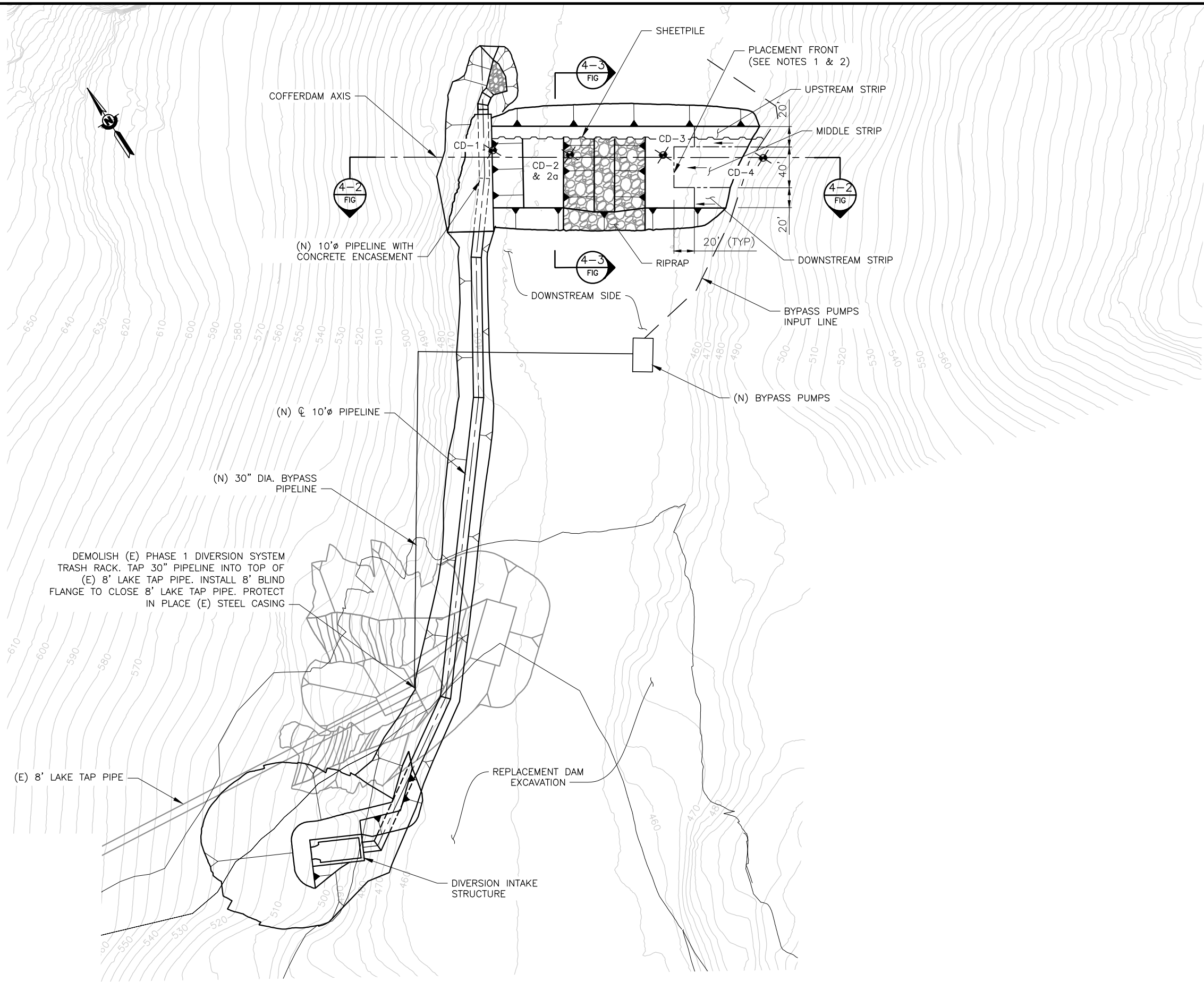
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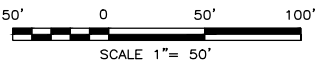
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Figures
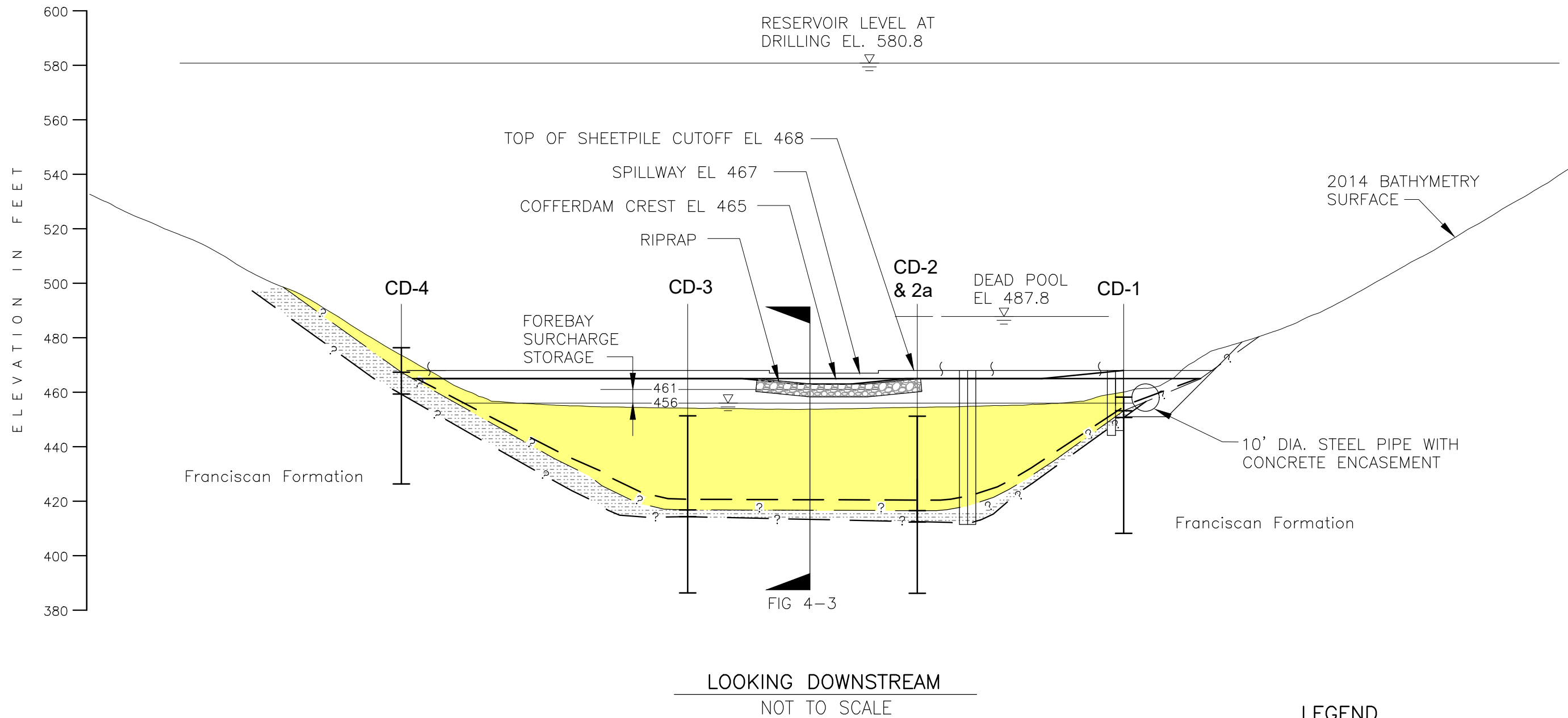
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NOTES:

1. PLACE DISPOSAL SITE FILL STARTING AT MIDDLE STRIP, GENERALLY MAINTAINING MIDDLE 40-FOOT STRIP 20-FEET AHEAD OF DOWNSTREAM STRIP AND UPSTREAM STRIP.

2. EXCAVATE MUDWAVE AND PLACE UPSTREAM OF COFFERDAM IF MUDWAVE INTERFERES WITH PLACEMENT OF DISPOSAL SITE FILL.

SHEETPILE

PLACEMENT FRONT (SEE NOTES 1 & 2)

UPSTREAM STRIP

COFFERDAM AXIS

MIDDLE STRIP

CD-1

CD-2 & 2a

CD-3

CD-4

20'

40'

20'

20' (TYP)

DOWNSTREAM STRIP

(N) 10'⌀ PIPELINE WITH CONCRETE ENCASEMENT

RIPRAP

DOWNSTREAM SIDE

BYPASS PUMPS INPUT LINE

(N) BYPASS PUMPS

(N) ℄ 10'⌀ PIPELINE

(N) 30" DIA. BYPASS PIPELINE

DEMOLISH (E) PHASE 1 DIVERSION SYSTEM TRASH RACK. TAP 30" PIPELINE INTO TOP OF (E) 8' LAKE TAP PIPE. INSTALL 8' BLIND FLANGE TO CLOSE 8' LAKE TAP PIPE. PROTECT IN PLACE (E) STEEL CASING

REPLACEMENT DAM EXCAVATION

(E) 8' LAKE TAP PIPE

DIVERSION INTAKE STRUCTURE

4-3 FIG

4-2 FIG

4-2 FIG

4-3 FIG

50' 0 50' 100'
SCALE 1" = 50'

PLAN
SCALE: 1"=50'

RESERVOIR LEVEL AT
DRILLING EL. 580.8

TOP OF SHEETPILE CUTOFF EL 468
SPILLWAY EL 467
COFFERDAM CREST EL 465
RIPRAP

2014 BATHYMETRY
SURFACE

CD-4

CD-3

CD-2
& 2a

DEAD POOL
EL 487.8

CD-1

FOREBAY
SURCHARGE
STORAGE

461
456

Franciscan Formation

10' DIA. STEEL PIPE WITH
CONCRETE ENCASEMENT

Franciscan Formation

FIG 4-3

ELEVATION IN FEET

600
580
560
540
520
500
480
460
440
420
400
380

LOOKING DOWNSTREAM
NOT TO SCALE

LEGEND

LAKEBED SEDIMENT OR
SOFT SOIL

Qal/Qc

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L:\DCS\Projects\Legacy\IE\AndersonSeisRetr_26818791\10000_CADD\10500_Temporary Working Documents & Sketches\Tuffy\90% DESIGN\Diversion Extension\Figure 4-3.dwg
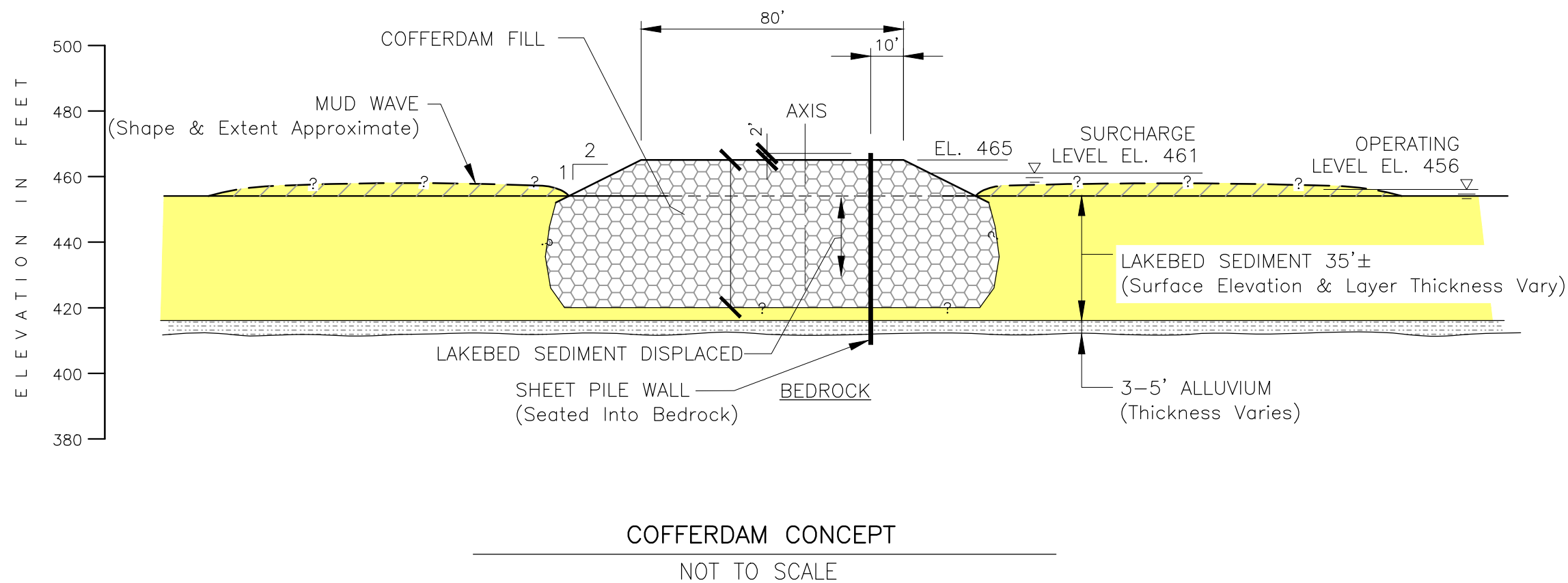
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COFFERDAM

SECTION ALONG COFFERDAM AXIS

Figure
4-2

COFFERDAM CONCEPT

NOT TO SCALE

Labels in the figure:

- COFFERDAM FILL
- MUD WAVE (Shape & Extent Approximate)
- ELEVATION IN FEET
- 500, 480, 460, 440, 420, 400, 380
- 80'
- 10'
- AXIS
- 2'
- EL. 465
- SURCHARGE LEVEL EL. 461
- OPERATING LEVEL EL. 456
- 1 2
- LAKEBED SEDIMENT 35'± (Surface Elevation & Layer Thickness Vary)
- LAKEBED SEDIMENT DISPLACED
- SHEET PILE WALL (Seated Into Bedrock)
- BEDROCK
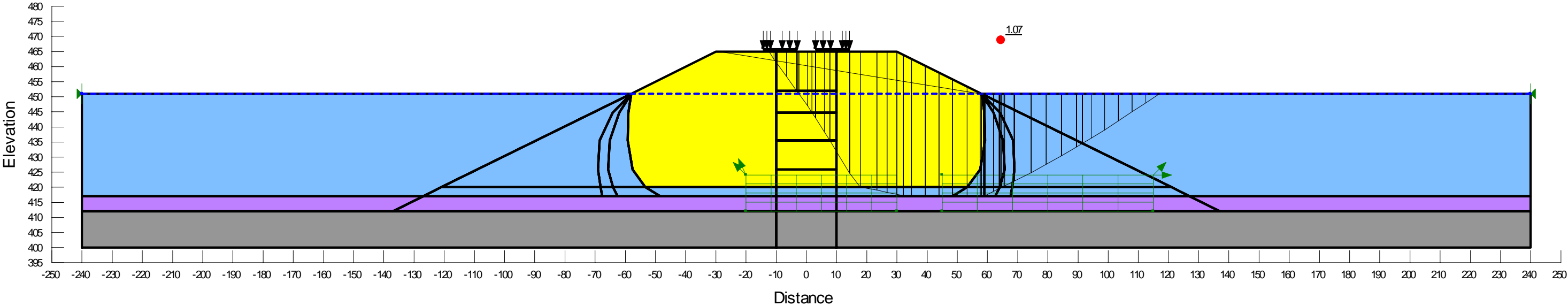- 3–5' ALLUVIUM (Thickness Varies)

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Project No. 26818793

Anderson Dam
Seismic Retrofit Project

COFFERDAM
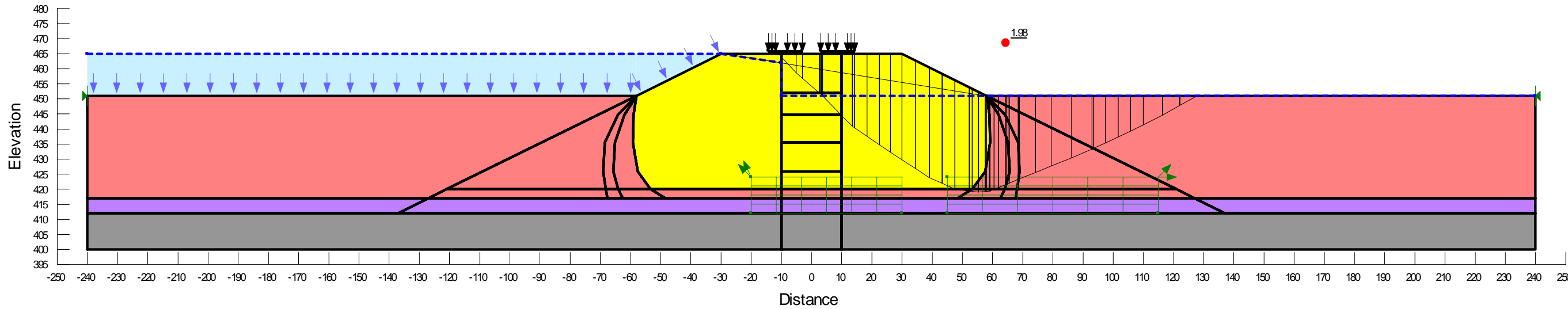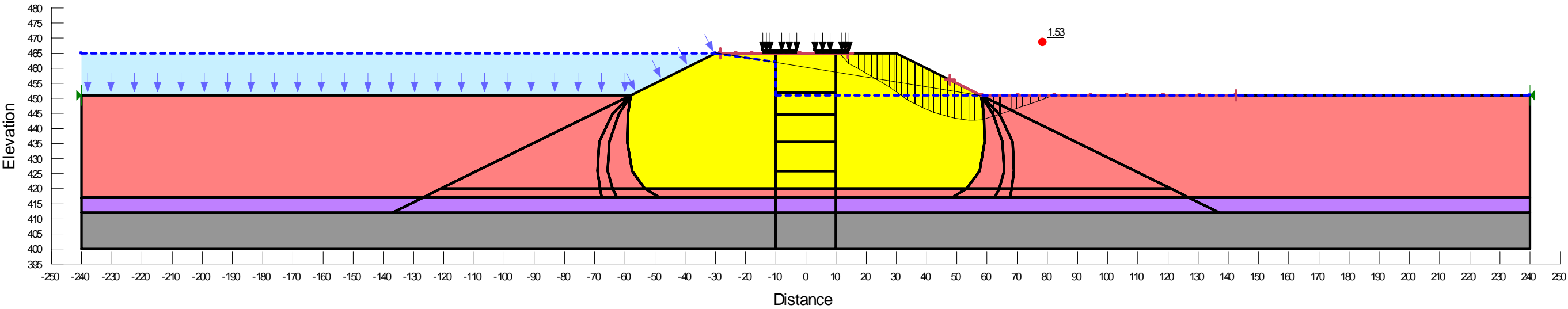TRANSVERSE SECTION

Figure
4–3

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)
🟨	Cofferdam Fill	120	0	30				
🟪	Alluvium/Colluvium	120	0	35				
⬜	Bedrock	150	0	38				
🟦	Lake Sediment (EOC)	100			0	8.65	10,000	451

Project No. 26818793	ANDERSON DAM SEISMIC RETROFIT PROJECT	End of construction stability	Figure
	URS		4-4

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Cofferdam Fill	120	0	30
	Alluvium/Colluvium	120	0	35
	Bedrock	150	0	38
	Lake Sediment (Drained)	100	0	27

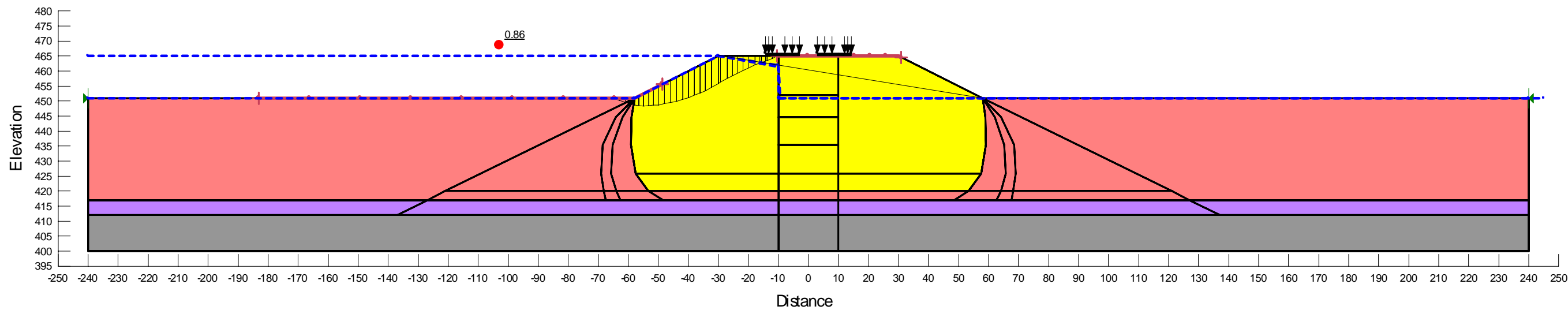| Project No. 26818793	ANDERSON DAM SEISMIC RETROFIT PROJECT	Long-term stability	Figure 4-5

| Color	Name	Unit Weight (pcf)	Tau/Sigma Ratio	Minimum Strength (psf)	Cohesion' (psf)	Phi' (°)	C-Datum (psf)	C-Rate of Change ((lbs/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)
(green)	Lake Sediment	100	0.23	0						
(yellow)	Cofferdam Fill	120			0	30				
(purple)	Alluvium/Colluvium	120			0	35				
(gray)	Bedrock	150			0	38				
(blue)	Lake Sediment (EOC)	100					0	8.65	10,000	451

Project No. 26818793

ANDERSON DAM
SEISMIC RETROFIT PROJECT

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Rapid loading stability

Figure

4-6

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Alluvium/Colluvium	Mohr-Coulomb	120	0	35	0	2
	Bedrock	Mohr-Coulomb	150	0	38	0	2
	Cofferdam Fill	Mohr-Coulomb	120	0	30	0	2
	Lake Sediment (Drained)	Mohr-Coulomb	100	0	27	0	2

	Project No. 26818793	ANDERSON DAM SEISMIC RETROFIT PROJECT	Rapid drawdown stability	Figure
		URS		4-7

Color	Name	Material Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Effective Cohesion (psf)	Effective Friction Angle (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line
	Alluvium/Colluvium	Mohr-Coulomb	120			0	35	0	0	1
	Bedrock	Mohr-Coulomb	150			0	38	0	0	1
	Cofferdam Fill	Mohr-Coulomb	120			0	30	0	30	1
	Lake Sediment	SHANSEP	100	0	0.23					1

$K_y = 0.07$

$K_y = 0.09$
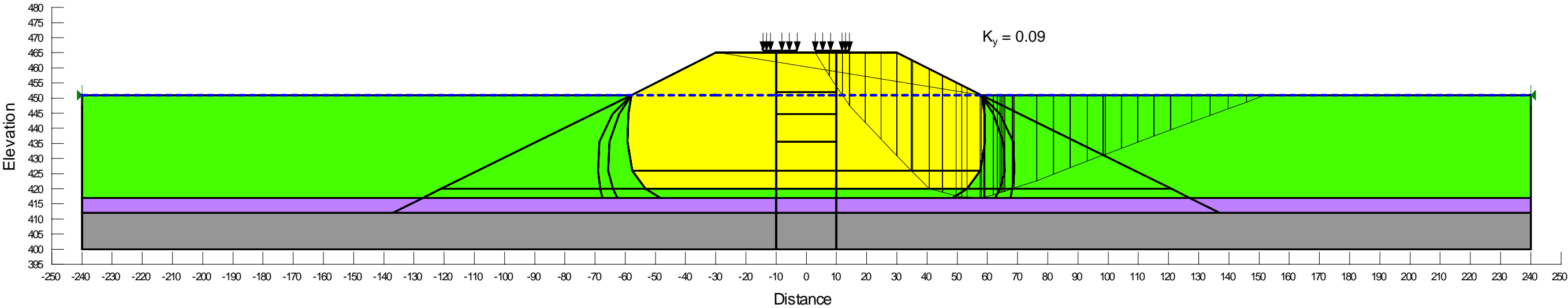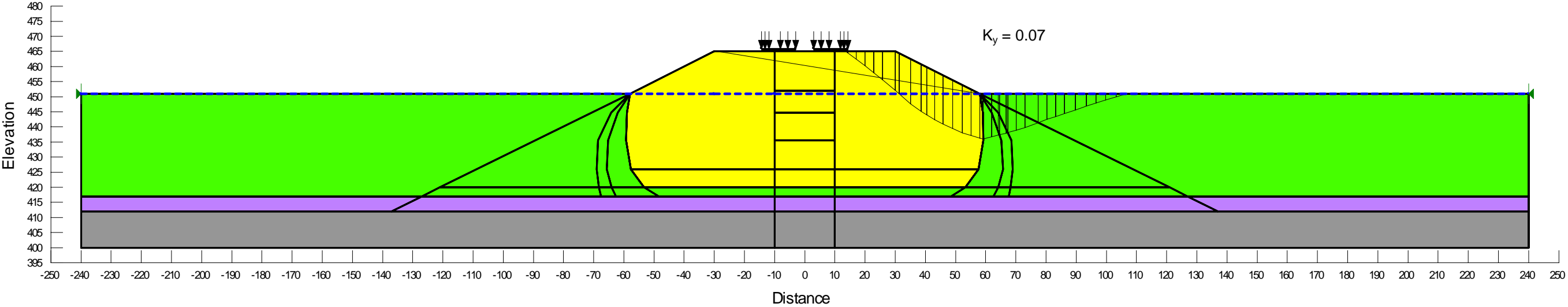
Project No. 26818793	ANDERSON DAM SEISMIC RETROFIT PROJECT	Pseudo-static stability	Figure 4-8

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Appendix A:

Triaxial Test Results and Undrained Shear Strength Ratio

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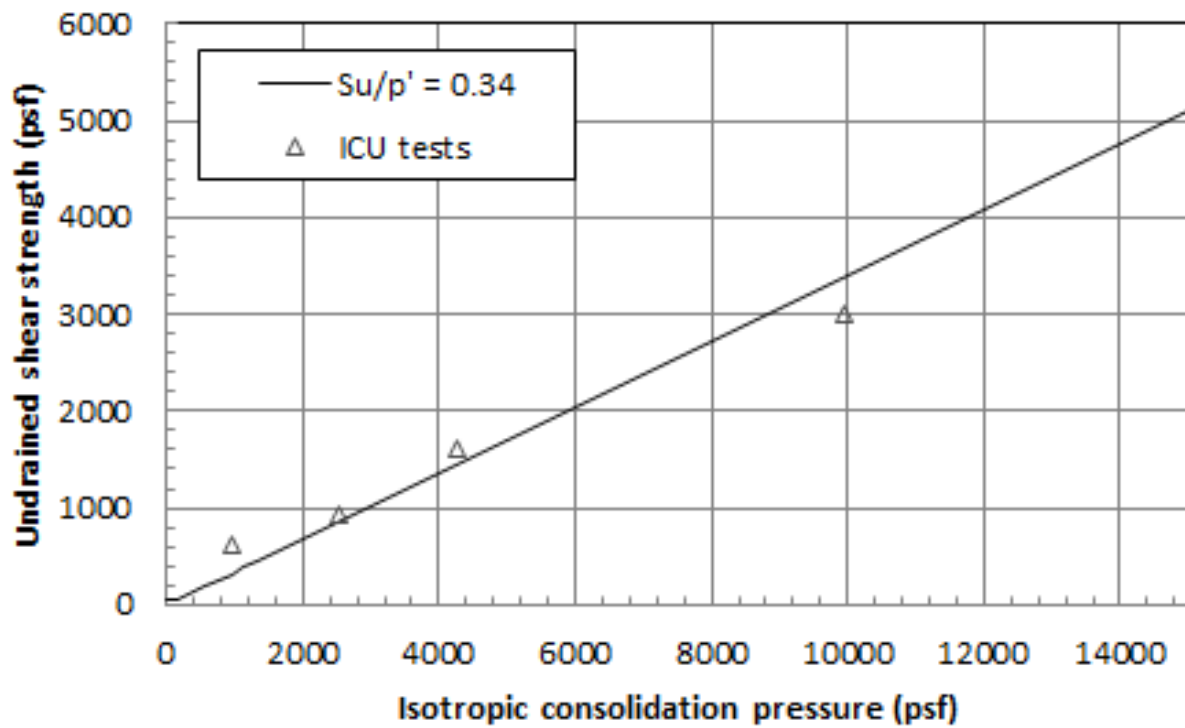
	Project No. 26818791	**ICU Triaxial Test Results and**	Figure
URS	Anderson Dam Seismic Retrofit Project	**Undrained Shear Strength Ratio**	A-1

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Appendix B:

Laboratory Test Data

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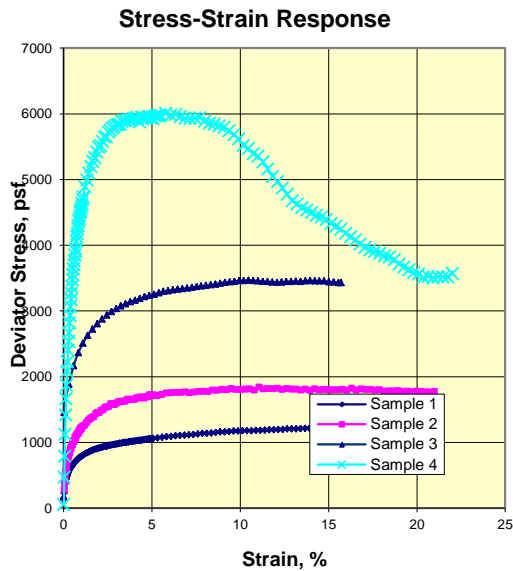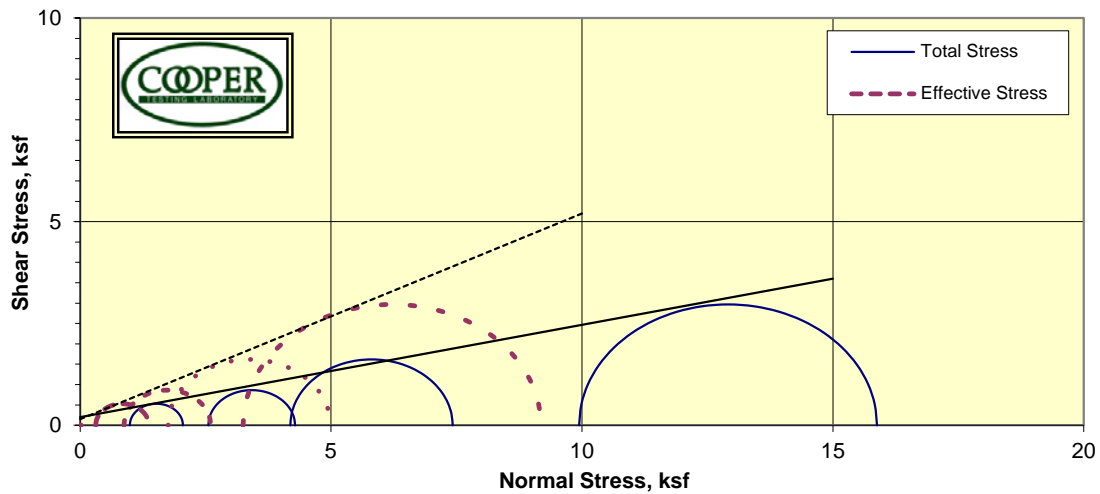*This page intentionally left blank.*

TABLE FOR CD AND LS BORINGS
SUMMARY OF SOIL LABORATORY DATA

Sample Information				USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	Sieve				Atterberg Limits			Gs	Other Tests
Boring Number	Sample Number	Depth, feet	Elevation, feet NAVD88				Gravel, %	Sand, %	<#200, %	<2μ, %	LL	PL	PI		
CD-2A	S-1	3.0-5.0	448.1	CH			0.0	0.7	99.3	44.4	62	30	32	2.75	
CD-2A	S-2	8.0-9.0	443.1	CH							65	32	33		
CD-2A	S-3	11.0-13.0	440.1	MH			0.0	0.4	99.6	34.9	52	29	23	2.77	CONS, TX-CU/TX-UU
CD-2A	S-4	18.0-20.0	433.1	CH							55	27	28		CONS, TX-CU/TX-UU
CD-2A	S-5	23.0-25.0	428.1	CH							58	28	30		TX-UU
CD-2A	S-6	30.0-32.0	421.1	MH			0.0	4.7	95.3	38.2	55	30	25	2.74	TX-CU/TX-UU
CD-5	S03-2	5.5-6.0	467.8	SM	14.4	112.1	33.8	40.9	25.3		52	40	12		
CD-5	S04-1	10.0-10.5	463.3	SC/Fm	18.4										
CD-6	S01	0-2.0	469.4	CH	56.5	64.0					64	29	35		
CD-6	S03-2	5.5-6.0	464.8	SM	13.2		9.9	48.9	41.2		49	46	3		
CD-7	S01	0-3.5	535.5	SC											SA
CD-7	S02-2	5.0-5.5	532.5	CL	8.5	116.0					24	16	8		
CD-7	S03-2	10.0-10.5	527.5	SC	6.0		28.0	43.9	28.1						
CD-7	S06	24.5-26.0	512.5	CL	11.4	119.0					30	19	11		
CD-8	S03-2	10.0-10.5	534.5	GC	4.7		29.6	23.0	47.4						
LS-11	S02	2.0-2.5	591.9	SM			5.4	72.3	22.3	9.4	NP	NP	NP		
LS-11	S04	5.5-6.5	588.4	CL			2.4	6.6	91.0		45	25	20		
LS-13	S02	3.0-5.0	535.1	CH							55	19	36		
LS-13	S04	8.0-10.0	530.1	CH							69	29	40		
LS-14	S02	3.0-5.0	527.2	CL							42	21	21		
LS-14	S06	12.5-14.5	517.9	ML			0.0	43.0	57.0		29	23	6		
LS-14	S09-2/3	19.5-20.5	511.4	ML							35	25	10		
LS-15	S02	0.5-2.5	513.8	CH			0.0	1.5	98.5		59	28	31		
LS-15	S06-2,3	10.5-11.5	504.5	SM			1.2	69.4	29.4	9.5	NP	NP	NP		
LS-15	S07-2	13.5-14.0	501.5	SC			29.9	48.8	21.3						
LS-15	S08	17.0-18.5	497.5	CH							63	26	37		
LS-16	S04	6.0-8.0	492.3	ML			0.0	13.5	86.5	16.9	40	26	14		
LS-16	S06-1/2	11.5-12.5	487.5	SM			0.0	69.2	30.8	9.9					
LS-17	S06	9.5-11.5	469.9	CH			0.0	0.1	99.9		78	27	51		
LS-18	S02	1.0-2.0	565.1	SM			28.0	56.8	15.2	5.1					
LS-18	S05	5.0-6.5	560.6	CL							46	23	23		
LS-20	S08	15.5-17.5	437.6	CH			0.0	0.5	99.5		51	22	29		
LS-20	S11	23.0-25.0	430.1	CH			0.0	0.1	99.9		68	29	39		

SCVWD Anderson Dam Seismic Retrofit
Morgan Hill, California

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Boring Number	Sample Number	Depth, feet	Elevation, feet NAVD88	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Gs	Other Tests
								Sieve				Atterberg Limits			
LS-22	S02	0-2.0	462.6	CH							80	30	50		
LS-22	S03	2.5-4.0	460.6	SM			0.0	80.5	19.5						
LS-23	S03	2.0-4.0	470.6	CL							33	22	11		
LS-23	S08-2/3	12.5-13.5	460.8	SM			2.8	63.3	33.9	11.9	NP	NP	NP		
LS-24	S02	0-2.0	480.6	CH							55	26	29		
LS-25	S02	0.5-2.0	493.3	CH			0.0	0.5	99.5	50.1	69	28	41		
LS-26	S02	0.5-2.0	502.8	CL							38	24	14		
LS-27	S01	0-0.5	518.7	CH							50	26	24		

NOTE: The laboratory tests were performed in general accordance with the following standards:

Water Content - ASTM Test Method D2216
Dry Unit Weight - ASTM Test Method D2937
Particle Size Distribution Analysis by Mechanical Sieving - ASTM Test Method D422 and D6913
Percent Passing No. 200 Sieve - ASTM Test Method D1140
Atterberg Limits - ASTM Test Method D4318
Specific Gravity of Soil - ASTM Test Method D854
Corrosivity Tests (CORR) - pH and Minimum Resistivity by Cal 643; Sulfate by Cal 417, Chloride by Cal 422
Crumb Test For Dispersive Soils (Crumb) - ASTM Test Method D6572
Hydraulic Conductivity (HC) - ASTM Test Method D5084
Pinhole Test For Dispersive Soils (PIN) - ASTM Test Method D4647
Slake Durability (Slake) - ASTM Test Method D4644
Consolidated Undrained Triaxial Compression Test (TX-CU) - ASTM Test Method D2850
Unconsolidated Undrained Triaxial Compression Test (TX-UU) - ASTM Test Method D4767
Unconfined Compressive Strengh of Soil (UCS) - ASTM Test Method D2166

SCVWD Anderson Dam Seismic Retrofit
Morgan Hill, California

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Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767

Shear Stress, ksf vs Normal Stress, ksf

- Total Stress
- Effective Stress

Stress-Strain Response

Deviator Stress, psf vs Strain, %

- Sample 1
- Sample 2
- Sample 3
- Sample 4

Sample:	1	2	3	4
MC, %	42.1	68.0	57.2	46.4
DD, pcf	77.3	59.6	64.3	68.9
Sat. %	95.2	99.1	94.5	85.7
Void Ratio	1.211	1.901	1.658	1.482
Diameter in	2.87	2.86	2.86	2.86
Height, in	5.86	6.00	6.00	6.00
Final				
MC, %	36.7	46.3	37.4	32.1
DD, pcf	85.2	75.7	84.4	91.0
Sat. %	100.0	100.0	100.0	100.0
Void Ratio	1.006	1.282	1.025	0.880
Diameter, in	2.75	2.60	2.58	2.61
Height, in	5.81	5.72	5.60	5.45
Cell, psi	66.6	77.6	94.6	129.4
BP, psi	59.8	59.9	65.5	60.4
Effective Stresses At:				
Strain, %	5.0	5.0	5.0	5.0
Deviator ksf	1.058	1.726	3.235	5.935
Excess PP	0.679	1.681	2.434	6.697
Sigma 1	1.369	2.602	4.991	9.182
Sigma 3	0.312	0.876	1.755	3.246
P, ksf	0.840	1.739	3.373	6.214
Q, ksf	0.529	0.863	1.618	2.968
Stress Ratio	4.394	2.971	2.843	2.828
Rate in/min	0.0005	0.0005	0.0000	0.0005

Job No.:	033-178	Date:	11/4/2014
Client:	URS		BY:DC
Project:	26818791		
Sample 1)	CD2-A;6 @ 30-32(Tip-7.5")	Gray Elastic SILT	
Sample 2)	CD-2A;3 @ 11-13(Tip-7")	Greenish Gray Elastic SILT	
Sample 3)	CD2-A;4 @ 18-20(Tip-4")	Gray Fat CLAY	
Sample 4)	CD2-A;6 @ 30-32(Tip-2")	Gray Elastic SILT	

Total C	0.2	ksf
Total phi	12.8	degrees
Eff. C	0.2	ksf
Eff. Phi	26.8	degrees ©

Unconsolidated-Undrained Triaxial Test
ASTM D2850

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Shear Stress, ksf vs. Total Normal Stress, ksf

Stress-Strain Curves

Legend:
- Sample 1
- Sample 2
- Sample 3
- Sample 4

Deviator Stress, ksf vs. Strain, %
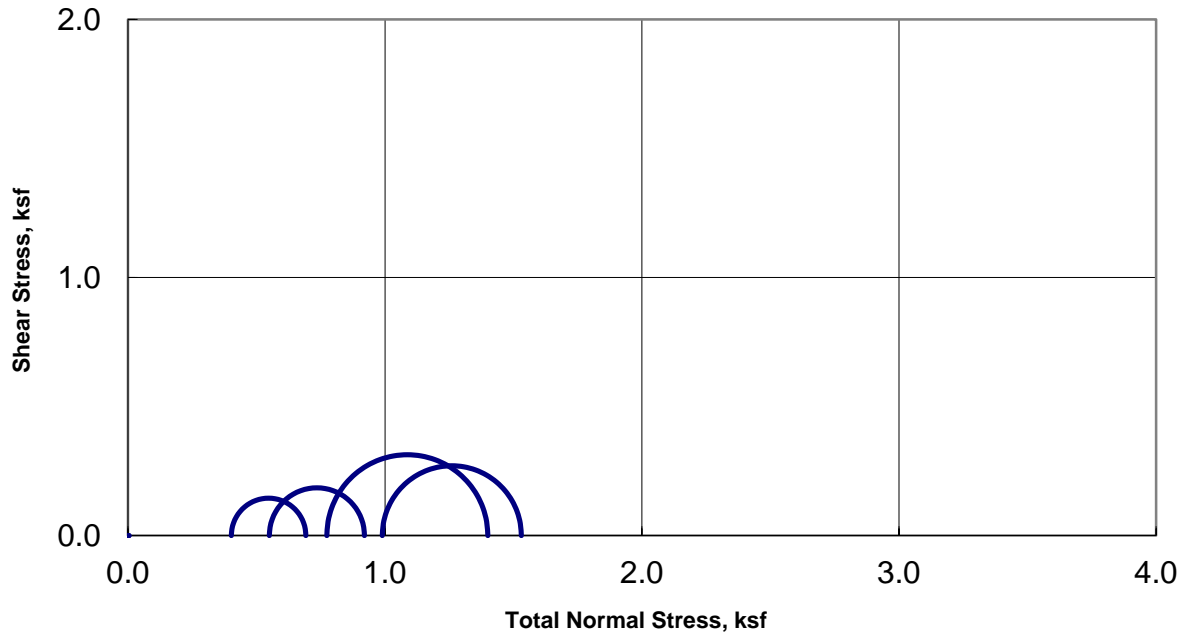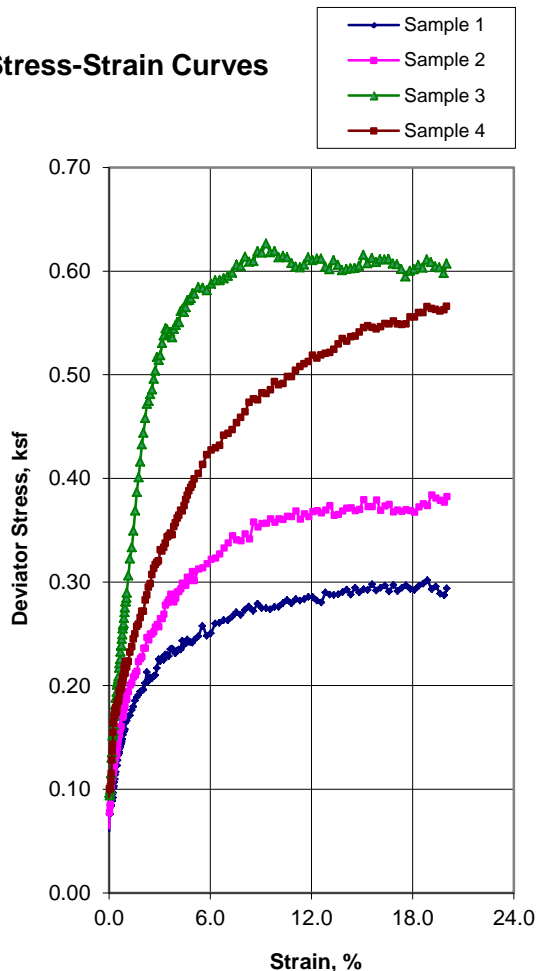
Sample Data	1	2	3	4
Moisture %	81.3	59.4	55.4	43.0
Dry Den,pcf	48.7	60.9	64.4	71.7
Void Ratio	2.551	1.767	1.617	1.387
Saturation %	88.3	90.8	92.5	84.9
Height in	5.80	5.85	5.87	5.84
Diameter in	2.87	2.87	2.87	2.87
Cell psi	2.8	3.8	5.4	6.9
Strain %	15.00	15.00	9.30	15.00
Deviator, ksf	0.290	0.370	0.627	0.541
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.058	0.059	0.059	0.058

Job No.:	**033-178**			
Client:	**URS**			
Project:	**26818791**			
Boring:	CD-2A	CD-2A	CD-2A	CD-2A
Sample:	3	4	5	6
Depth ft:	11-13(Tip-14")	18-20	23-25(Tip-2")	30-32(Tip-16.5")

Visual Soil Description	
Sample #	
1	Greenish Gray Elastic SILT
2	Gray Fat CLAY
3	Dark Greenish Gray Fat CLAY
4	Gray Elastic SILT
Remarks:	

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Consolidation Test
ASTM D2435

Job No.:	033-178	**Boring:**	CD-2A	**Run By:**	MD
Client:	URS	**Sample:**	4	**Reduced:**	PJ
Project:	26818791	**Depth, ft.:**	18-20(Tip-2.5")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Fat CLAY			**Date:**	10/23/2014

Void Ratio - Log P Curve

Void Ratio (e) vs Effective Stress, psf
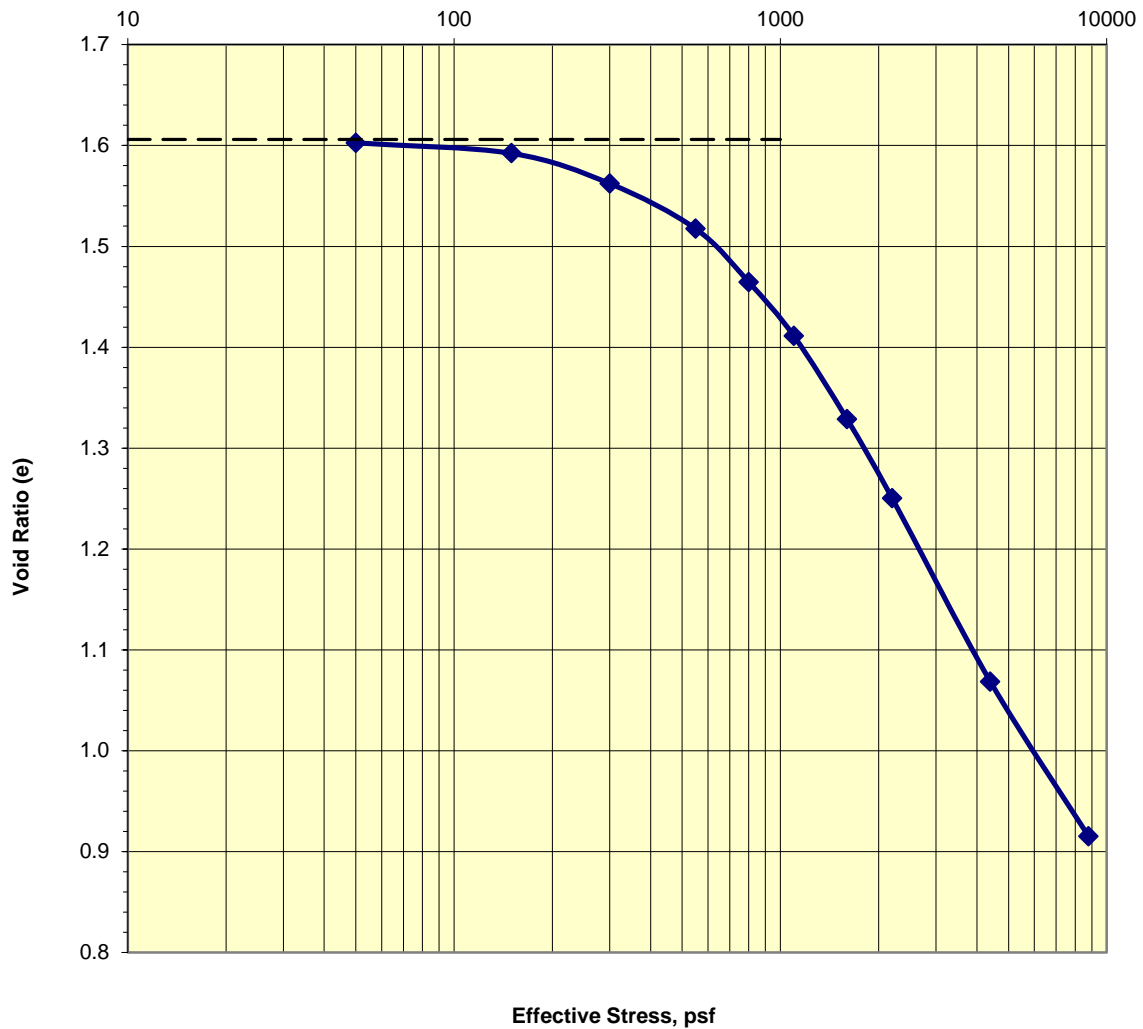
Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	57.1	39.4
Dry Density, pcf:	64.7	81.6
Void Ratio:	1.606	1.065
% Saturation:	96.0	100.0

Remarks:

Consolidation Test
ASTM D2435

Job No.:	033-178	**Boring:**	CD-2A	**Run By:**	MD
Client:	URS	**Sample:**	3	**Reduced:**	PJ
Project:	26818791	**Depth, ft.:**	11-13(Tip-5")	**Checked:**	PJ/DC
Soil Type:	Greenish Gray Elastic SILT			**Date:**	10/23/2014

Void Ratio -Log P Curve

Effective Stress, psf
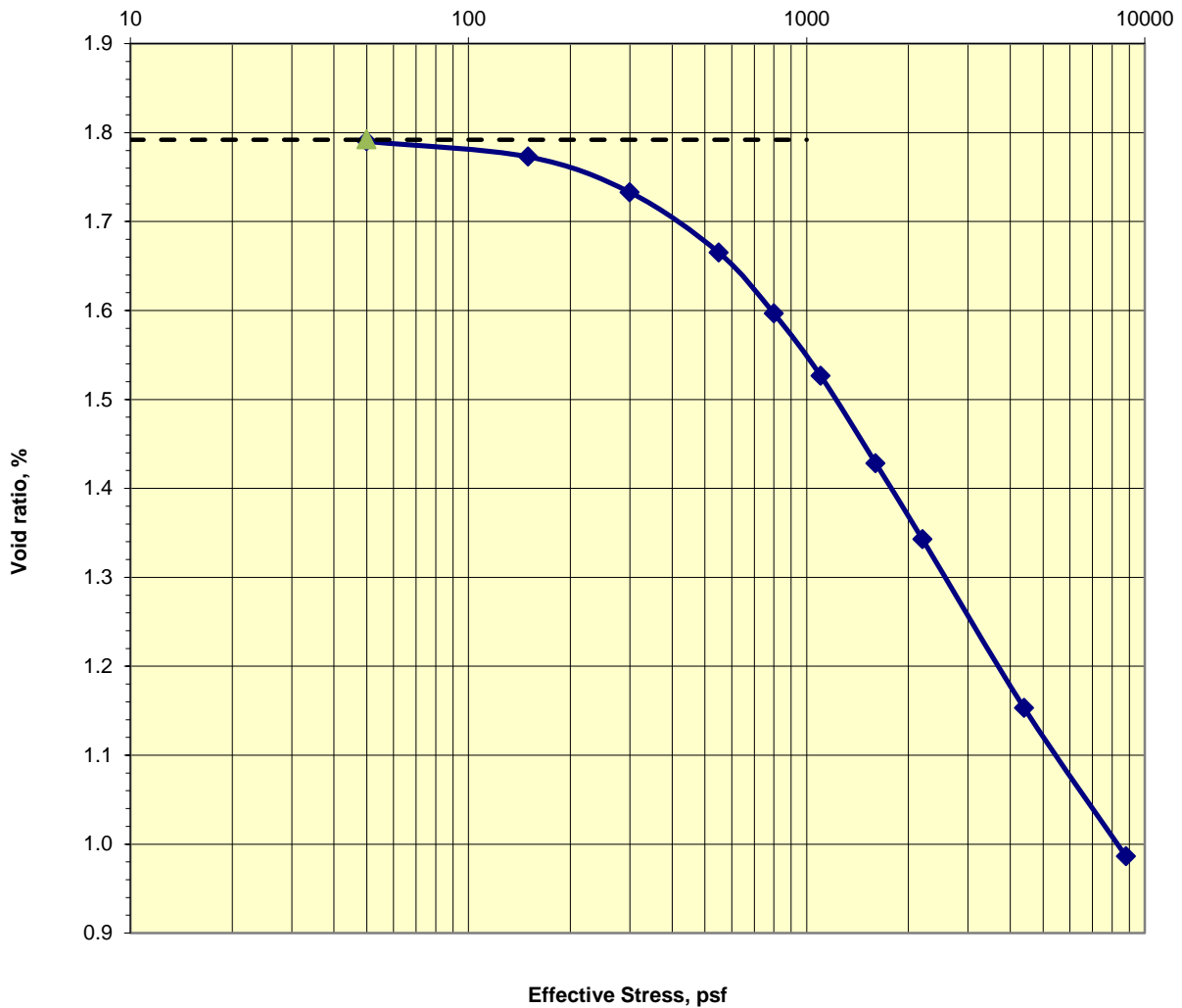
Measured Gs	2.77	**Initial**	**Final**	**Remarks**:
Moisture %:		61.2	40.0	
Dry Density, pcf:		61.9	82.0	
Void Ratio:		1.792	1.109	
% Saturation:		94.5	100.0	

LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils

PLASTICITY INDEX vs LIQUID LIMIT chart (CL or OL, CH or OH, ML or OL, MH or OH, CL-ML zones)

WATER CONTENT vs NUMBER OF BLOWS chart
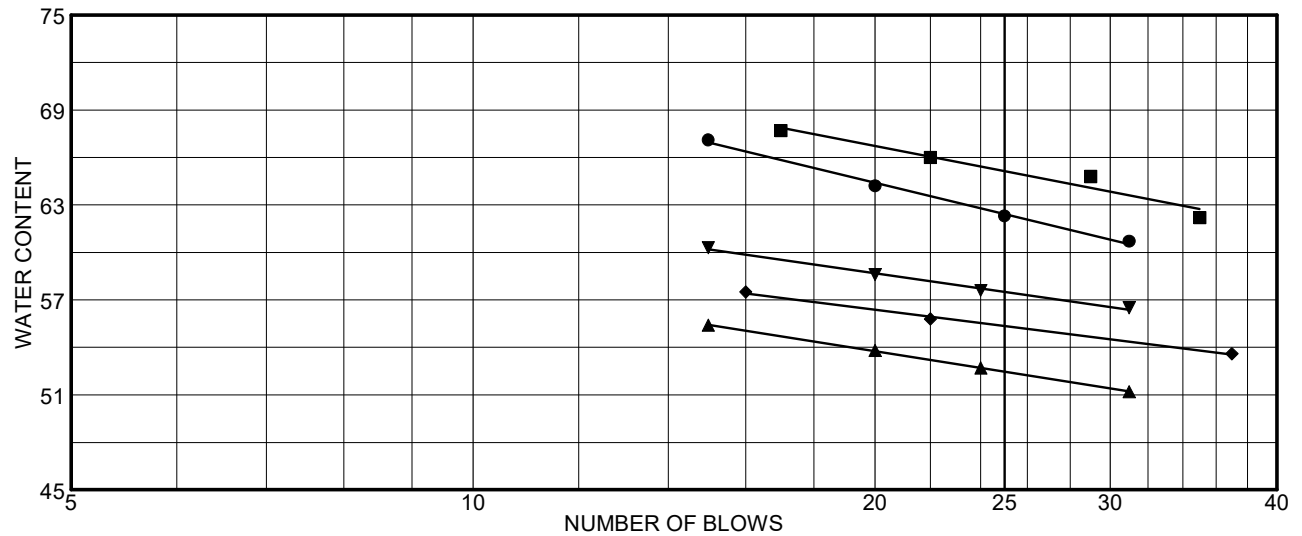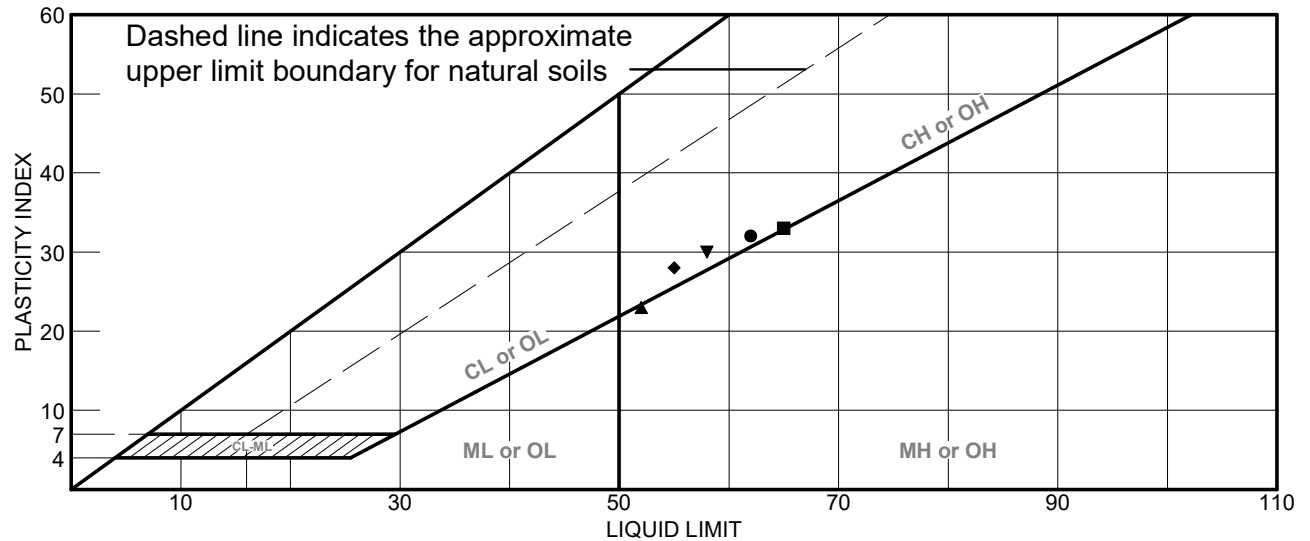
	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Greenish Gray Fat CLAY w/ organics	62	30	32	99.9	99.3	CH
■	Greenish Gray Fat CLAY	65	32	33			
▲	Greenish Gray Elastic SILT	52	29	23	99.9	99.6	MH
◆	Greenish Gray Fat CLAY	55	27	28			
▼	Dark Greenish Gray Fat CLAY	58	28	30			

Project No. 033-178   **Client:** URS

Project: Anderson Dam Seismic Retrofit Project - 26818791

●**Source:** CD-2A	**Sample No.:** 1	**Elev./Depth:** 3-5(Tip-3")
■**Source:** CD-2A	**Sample No.:** 2	**Elev./Depth:** 8-9'
▲**Source:** CD-2A	**Sample No.:** 3	**Elev./Depth:** 11-13'
◆**Source:** CD-2A	**Sample No.:** 4	**Elev./Depth:** 18-20'
▼**Source:** CD-2A	**Sample No.:** 5	**Elev./Depth:** 23-25(Tip-2")

Remarks:
●
■
▲
◆
▼

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils
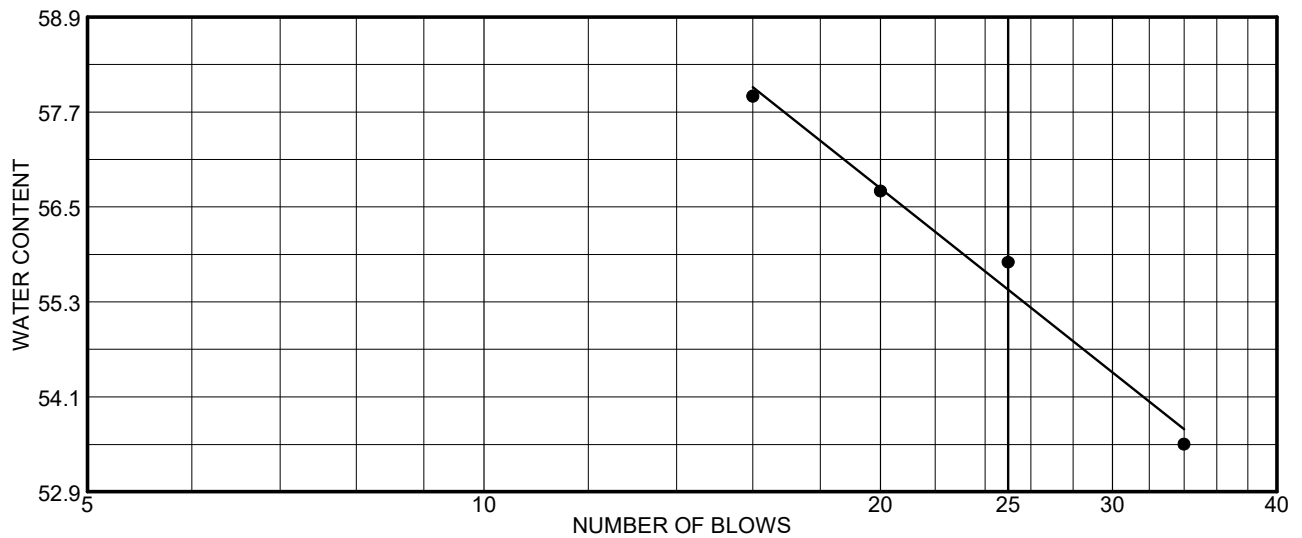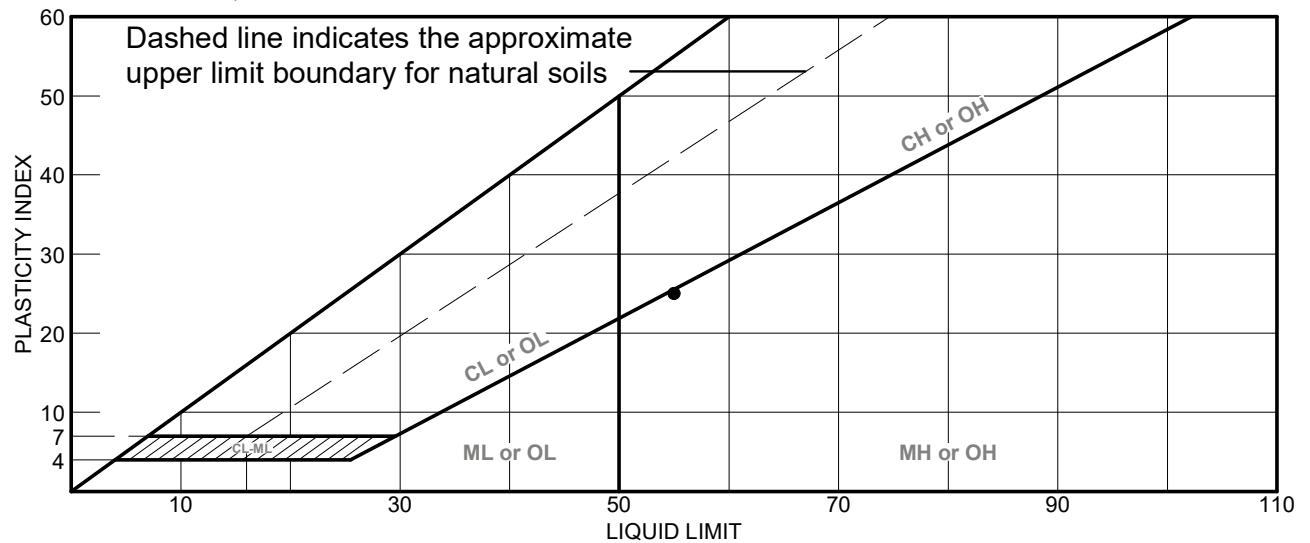
MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
Gray Elastic SILT	55	30	25	99.7	95.3	MH

Project No. 033-178   **Client:** URS

Project: Anderson Dam Seismic Retrofit Project - 26818791

Source: CD-2A   **Sample No.:** 6   **Elev./Depth:** 30-32'

Remarks:

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report
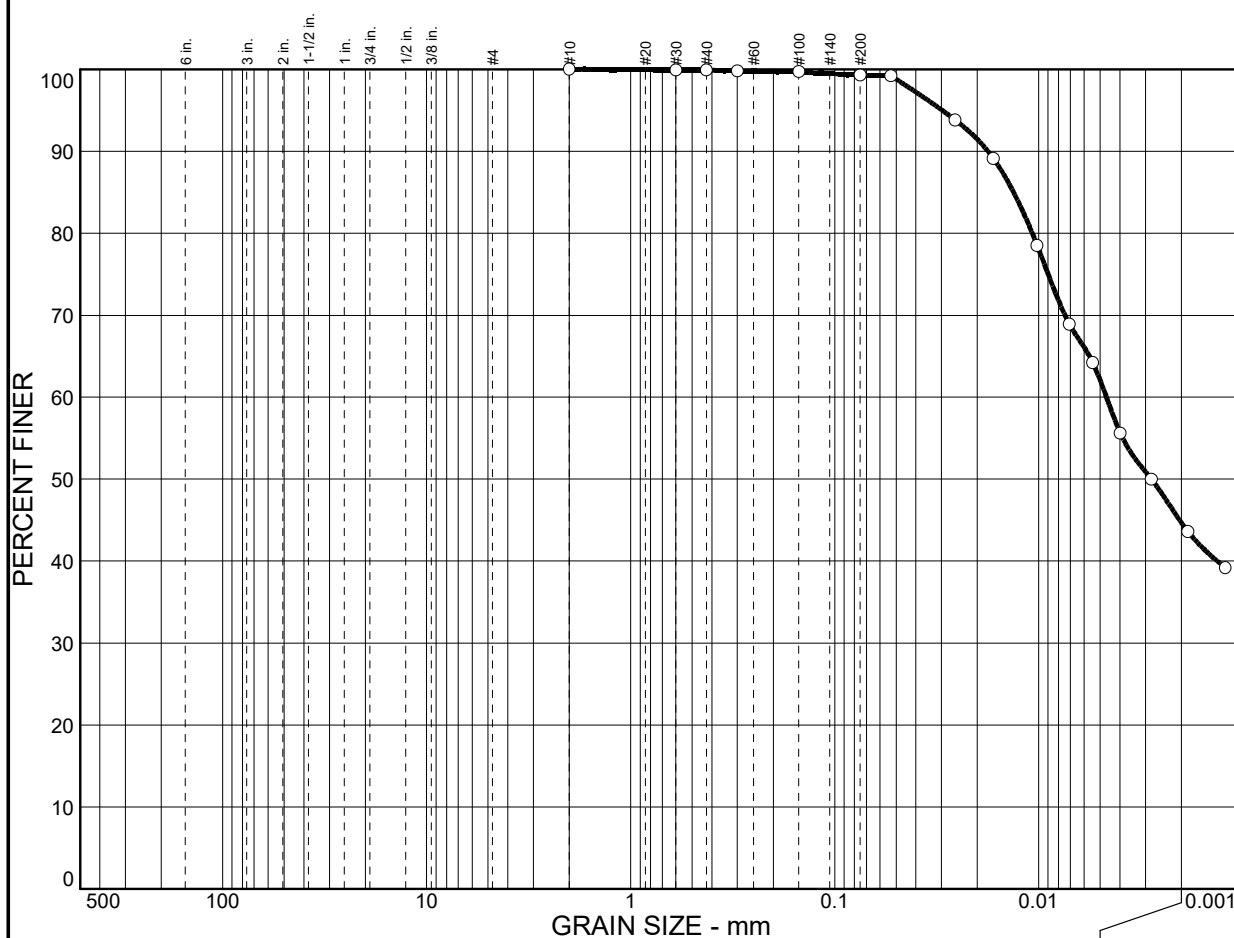
% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	0.7	54.6	44.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	99.9		
#40	99.9		
#50	99.8		
#100	99.7		
#200	99.3		
#270	99.2		
0.0257 mm.	93.8		
0.0167 mm.	89.1		
0.0102 mm.	78.5		
0.0071 mm.	68.9		
0.0054 mm.	64.2		
0.0040 mm.	55.6		
0.0028 mm.	50.0		
0.0019 mm.	43.6		
0.0012 mm.	39.2		

(no specification provided)

Soil Description

Dark Greenish Gray Fat CLAY w/ organics

Atterberg Limits

PL= 30   LL= 62   PI= 32

Coefficients

D_{85}= 0.0133 D_{60}= 0.0047 D_{50}= 0.0028
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CH   AASHTO=

Remarks

Sample No.: 1   **Source of Sample:** CD-2A   **Date:** 11/4/14
Location: **Elev./Depth:** 3-5(Tip-3")

COOPER TESTING LABORATORY

Client: URS
Project: Anderson Dam Seismic Retrofit Project - 26818791

Project No: 033-178   **Figure**

Particle Size Distribution Report

% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	0.4	64.4	35.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	99.9		
#40	99.9		
#50	99.8		
#100	99.8		
#200	99.6		
#270	99.5		
0.0265 mm.	87.1		
0.0175 mm.	78.0		
0.0107 mm.	63.8		
0.0078 mm.	57.8		
0.0056 mm.	51.2		
0.0040 mm.	45.6		
0.0029 mm.	38.8		
0.0021 mm.	35.5		
0.0012 mm.	28.3		

* (no specification provided)

Soil Description
Greenish Gray Elastic SILT

Atterberg Limits
PL= 29   LL= 52   PI= 23

Coefficients
D_{85}= 0.0238 D_{60}= 0.0088 D_{50}= 0.0052

D_{30}= 0.0014 D_{15}= D_{10}=

C_u= C_c=

Classification
USCS= MH   AASHTO=

Remarks

Sample No.: 3   **Source of Sample:** CD-2A   **Date:** 10/16/14

Location: **Elev./Depth:** 11-13'

COOPER TESTING LABORATORY

Client: URS

Project: Anderson Dam Seismic Retrofit Project - 26818791

Project No: 033-178   **Figure**

Particle Size Distribution Report

% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	4.7	57.1	38.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	99.8		
#40	99.7		
#50	99.5		
#100	98.8		
#200	95.3		
#270	90.9		
0.0289 mm.	76.2		
0.0189 mm.	67.3		
0.0112 mm.	60.4		
0.0081 mm.	55.7		
0.0059 mm.	46.2		
0.0042 mm.	44.1		
0.0029 mm.	42.1		
0.0021 mm.	38.7		
0.0013 mm.	33.3		

* (no specification provided)

Soil Description

Gray Elastic SILT

Atterberg Limits

PL= 30   LL= 55   PI= 25

Coefficients

D_{85}= 0.0406 D_{60}= 0.0107 D_{50}= 0.0067
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH   AASHTO=

Remarks

Sample No.: 6   **Source of Sample:** CD-2A   **Date:** 10/27/14
Location: **Elev./Depth:** 30-32'

COOPER TESTING LABORATORY

Client: URS

Project: Anderson Dam Seismic Retrofit Project - 26818791

Project No: 033-178   **Figure**

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Appendix C:

Bypass Pumping

URS

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AECOM

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To:	Project Files	Pages:	8
Through:	John Roadifer, PE		
Subject:	Cofferdam Bypass Pumping		
Project	Anderson Dam Seismic Retrofit Project		
From:	Idit Zarci, PE		
Date:	November 1, 2021		

This memorandum summarizes the initial analysis of the Anderson Dam Seismic Retrofit Project cofferdam pumping requirements.

1.0   Background

The Anderson Dam seismic retrofit requires a temporary cofferdam and pumping during the first and fifth embankment construction seasons (ADSRP Year 2 and 6). The cofferdam base, crest, and top of sheet pile elevations are approximately 455 ft, 465 ft, and 467 ft, respectively. The maximum spring shoulder season capture rate is assumed to be 30 cfs or 13,465 gpm. The maximum summer capture rate is approximately 5 cfs or 2,244 gpm. The discharge location during the first embankment season is the Stage 1 Diversion System Intake Structure approximately 950 ft downstream with an invert elevation of approximately 450 ft. The discharge location during the fifth embankment season is the High-Level Outlet Works (HLOW) Intake Structure approximately 1,200 ft downstream with an invert elevation of 528 ft. The maximum static head requirement for a pump would be 73 ft when pumping to the HLOW.

2.0   Discussion

2.1   Pump Models

The 30 cfs pumping system will be required for two spring shoulder seasons and summers. Because of the temporary nature, the pumps would be rented from a rental venue like Rain for Rent, Sunbelt Rentals, or United Rentals. For this discussion, Rain for Rent pumps will be used. From Rain for Rent's 2016 Product Handbook, there are forty-nine (49) pump models (centrifugal, axial, and submersible) that could apply as shown in Appendix A Table 1. All the pumps can throttle and are either diesel driven directly or indirectly through diesel gensets. Of the available pumps, thirty-four (34) can be eliminated because of the number of pumps required to convey 30 cfs. Too many (more than four) would be logistically challenging and require more laydown area than is available. One pump provides inadequate coverage and redundancy and requires higher net positive suction head (NPSHr) than is available. Thus, two to four pumps are assumed to be optimal. Furthermore, submersible pumps and non-self-contained pumps requiring additional fuel tanks and gensets are eliminated due to additional complication to deploy compared to self-contained pumps. Data for the most applicable pump from the Rent for Rent applicable catalog is shown in Table 1.

Table 1: Rain for Rent applicable pumps.

n	Model	Q, max (gpm)	H, max (ft)	H, q_des (ft)	Dia (in)		Fuel		Photo
					Suction	Discharge	Storage (gal)	Economy (gph)	
1	DV325c	8,500	220	110	14	12	340	12.5	

2.2   Suction and Discharge Pipeline Diameters

Per Hydraulic Institute Standards, a pumped system design should consider suction velocities between 3.0 to 5.0 ft per second and discharge velocities between 4.0 to 8.0 ft per second. From Appendix A Table 2, the minimum suction and discharge pumping diameters are 24" and 30", respectively.

2.3   Pump Placement

Pump placement is critical for pump performance. Two criteria affected by placement are net positive suction head available (NPSHa)/net positive suction head required (NPSHr) and total dynamic head (TDH).

2.3.1   NPSHA AND NPSHR

The pump NPSHr represents the minimum suction head required for the pump to operate without cavitation and is typically determined by the pump manufacturer with empirical testing. TDH represents the total height that a fluid is to be pumped including friction losses of the system.

There are two potential locations for pump placement: on or near the cofferdam crest or downstream of the cofferdam. Figure 1 illustrates the pumps located on or near the cofferdam crest at elevation 465 ft and the pump centerline 2 ft above the cofferdam crest at elevation 467 ft. Figure 2 illustrates the pumps downstream of the cofferdam with the pump centerline at elevation 457 ft.

For the pumps located on or near the cofferdam crest, the NPSHa is approximately 16.3 ft (see NPSHa calculation table in Appendix 1). For the pumps located at the downstream location, the NPSHa is approximately 25.3 ft.

For the DV325c model, the NPSHr is 22 ft (**Error! Reference source not found.**). Therefore, locating the DV325c model at the cofferdam crest is not feasible as the NPSHr (22 ft) is greater than the NPSHa (16.3 ft). As such, the maximum water elevation that can be drawdown based on NPSHr/NPSHa is approximately 453.7 ft or [457 ft - (25.3 ft-22 ft)], which means the entire cofferdam can be drawn down.

Based on the NPSHa and NPSHr, placement of the pumps at the crest of the cofferdam is not feasible. The pumps would need to be located at the downstream location with a centerline elevation of approximately 457 ft.

2.3.2 TOTAL DYNAMIC HEAD (TDH)

Stage 1 Diversion

The cofferdam water elevation will range from 455 ft to 465 ft and the downstream Stage 1 diversion intake is located at elevation 450 ft. Therefore, gravity flow from the cofferdam to the Stage 1 diversion intake is feasible. However, because the bypass piping would be routed over the sheet pile wall of the cofferdam, a localized high point is created at the top of the sheet pile wall and as such, gravity flow cannot occur unless the bypass piping is primed to create a siphon.

A vacuum pump can be installed, instead of the DV325c model, which would be operated to remove the air from the suction piping creating a vacuum and allowing the water to flow over the top of the sheet pile. Once the pipeline has been primed, flow can occur by gravity and would be self-sustaining provided the pipeline remains primed.

HLOW Intake Diversion

Pumping to the HLOW intake represent the maximum total dynamic head requirements of 79.8 ft (El. 528 – El. 450 + 1.8 ft of frictional losses).

3.0 Recommendations

The cofferdam bypass pumping will be designed, furnished, and installed by the Contractor. However, from this initial analysis, the following is recommended based on the Rain for Rent Product Handbook:

1. Install two vacuum pumps located at the cofferdam crest for diversions to the Stage 1 Diversion Intake. The pumps would be used to prime the bypass pipingfor gravity flow at the 30 cfs design flow rate. A valve can be installed to stop the siphon if needed.
2. Utilize two DV325c model pumps for a pumping capacity of 30 cfs for diversions to the HLOW Intake.
    a. Pumps would be located downstream of the cofferdam with a centerline elevation at approximately 457 ft.
3. The suction pipeline should include the following characteristics:
    a. Rigid material with solvent welded joints i.e. PVC, HDPE to minimize potential to draw in air at joints; and
    b. A minimum of 24" diameter
4. The discharge and manifold pipeline should include the following characteristics:
    a. Rigid material i.e. PVC, HDPE, ductile iron, etc.
    b. A minimum of 30" diameter; and
    c. A check valve on pump discharge pipe
5. Valves for operation and isolation
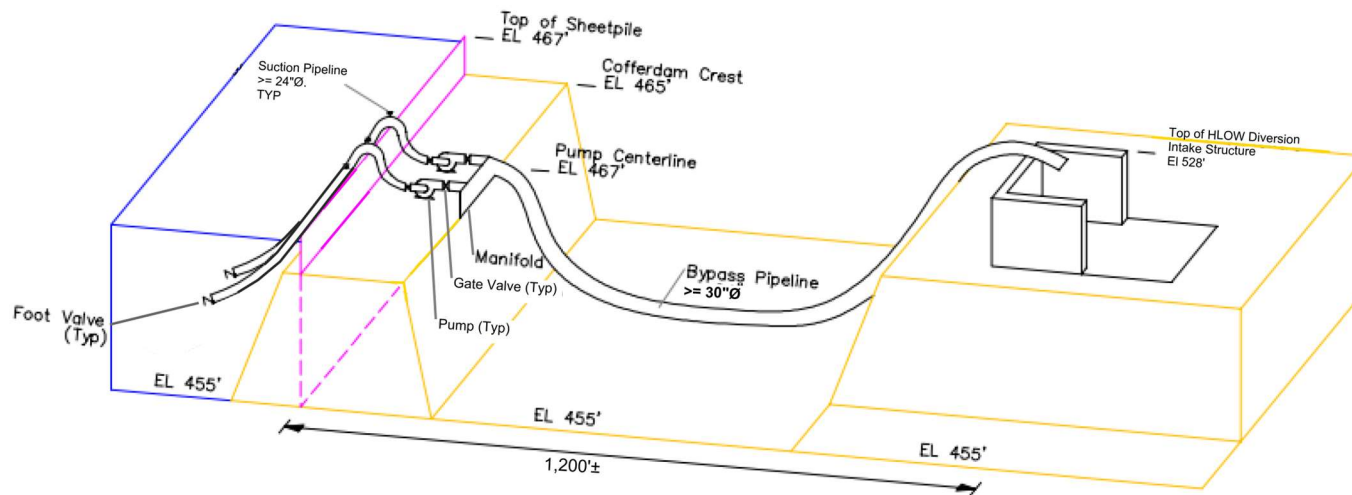
Figure 1. Pump placement on the cofferdam crest at elevation 465 ft.
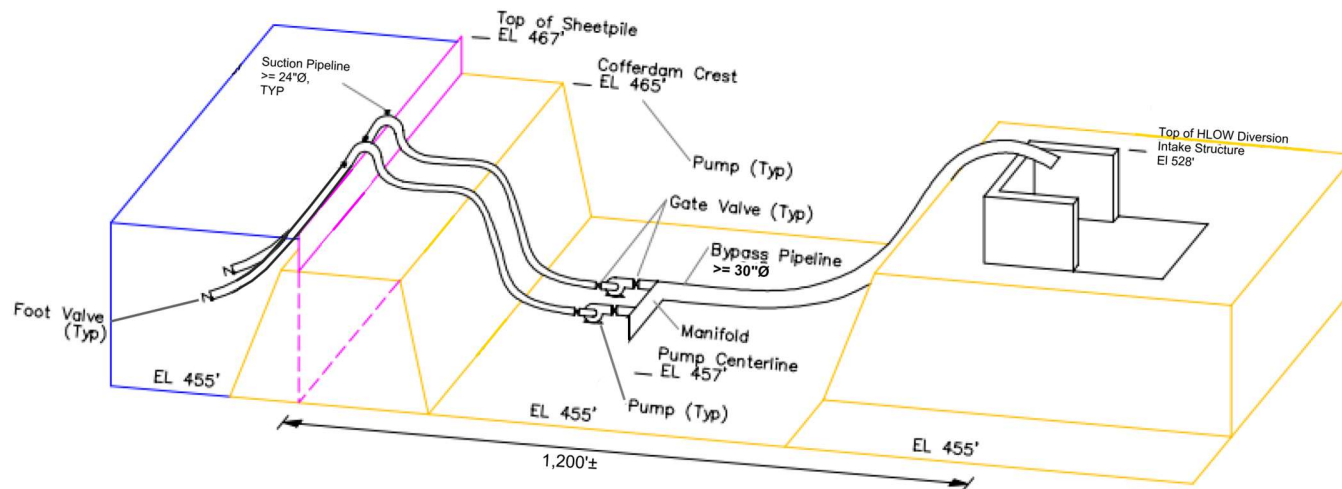
Figure 2. Pump placement at downstream location at elevation 455 ft.

Appendix A

Q, design=   13,465 gpm   30.0 cfs   19.39 mgd

Table 1. Rain for Rent pump models.

n	Model	Q, max (gpm)	H, max (ft)	Suction Lift (ft)	Dia (in) Suction	Discharge	# of Pump	Vel (ft/s) Suction	Discharge	Elimin-ation
1	DV80	500	135	28	3	3	27	22.6	22.6	# of Pumps
2	DV80c	880	125	28	4	3	16	21.5	38.2	# of Pumps
3	DV80m	630	94	28	3	3	22	27.8	27.8	# of Pumps
4	DV100	790	115	28	4	4	18	19.1	19.1	# of Pumps
5	DV100c	1450	165	28	6	4	10	15.3	34.4	# of Pumps
6	DV150	2200	157	28	6	6	7	21.8	21.8	# of Pumps
7	DV150i	2750	195	28	6	6	5	30.6	30.6	# of Pumps
8	DV175c	2900	295	28	8	6	5	17.2	30.6	# of Pumps
9	DV200	2775	155	28	8	8	5	17.2	17.2	# of Pumps
10	DV200c	4600	260	28	12	5	3	12.7	73.3	v_discharge
11	DV300	5000	115	28	12	10	3	12.7	18.3	
12	DV300i	6900	197	28	12	12	2	19.1	19.1	NPSHr
13	DV325c	8500	220	28	18	12	2	8.5	19.1	
14	DV350c	13500	180	28	14	14	1	28.1	28.1	# of Pumps
15	DV400c	16000	200	28	18	16	1	17.0	21.5	# of Pumps
16	DV600c	28000	96	28	30	24	1	6.1	9.5	# of Pumps
17	HH80	450	320	28	3	3	30	20.4	20.4	# of Pumps
18	HH80c	450	360	28	3	3	30	20.4	20.4	# of Pumps
19	HH125	900	370	28	6	4	15	10.2	22.9	# of Pumps
20	HH125c	1525	355	28	6	4	9	17.0	38.2	# of Pumps
21	HH150	2300	319	28	8	6	6	14.3	25.5	# of Pumps
22	HH160i	2800	460	28	8	6	5	17.2	30.6	# of Pumps
23	HH200i	4100	370	28	8	8	4	21.5	21.5	v_discharge
24	HH225c	5400	405	28	12	8	3	12.7	28.6	v_discharge
25	HH300c	6800	415	28	12	10	2	19.1	27.5	v_discharge
26	RL200	975	277	28	8	8	14	6.1	6.1	# of Pumps
27	VP150	2200	107	28	6	6	7	21.8	21.8	# of Pumps
28	VMX150	2300	157	28	6	6	6	25.5	25.5	# of Pumps
29	VP500	22000	120	28	24	20	1	9.5	13.8	# of Pumps
30	HD600	19000	47	-40	30	24	1	6.1	9.5	# of Pumps
31	ES600	17500	24	-20	TBD	24	1		9.5	# of Pumps
32	FP900	48500	35	TBD	TBD	36	1		4.2	# of Pumps
33	FP1050	68500	27	TBD	TBD	42	1		3.1	# of Pumps
34	XH100	1250	605	28	6	4	11	13.9	31.3	# of Pumps
35	XH150	2350	605	28	8	6	6	14.3	25.5	# of Pumps
36	XH125	1600	950	TBD	6	5	9	17.0	24.4	# of Pumps
37	ST4	850	110	Submersible	TBD	4	16		21.5	Complexity
38	S6T	1600	110	Submersible	TBD	6	9		17.0	Complexity
39	S6TDI	1600	110	Submersible	TBD	6	9		17.0	Complexity
40	S4CSL	740	100	Submersible	TBD	4	19		18.1	Complexity
41	S4THL	1000	215	Submersible	TBD	4	14		24.6	Complexity
42	S6200	3500	220	Submersible	TBD	8	4		21.5	Complexity
43	S6300	8500	110	Submersible	TBD	12	2		19.1	Complexity
44	3HA	1100	475	Low NPSHr	6	3	13	11.8	47.0	# of Pumps
45	4HH	1600	420	Low NPSHr	6	4	9	17.0	38.2	# of Pumps
46	3RB	800	260	Low NPSHr	5	3	17	12.9	36.0	# of Pumps
47	4RB	1600	250	Low NPSHr	6	4	9	17.0	38.2	# of Pumps
48	5RB	3000	370	Low NPSHr	8	5	5	17.2	44.0	# of Pumps
49	6RB	4500	300	Low NPSHr	10	6	3	18.3	50.9	v_discharge

L,discharge =	700 ft		ε =	1.80E-03 ft	
L,suction =	500				
ν =	1.21E-05 ft^2/s		$h_{f,\ Minor}$ =	30 %	

Table 2. Discharge pipeline velocities at Q, design.

Dia (in)	Area, A		V (ft/S)	Re	f	$h_{f,\ Major}$	$h_{f,\ Minor}$	S.F	$h_{f,\ Total}$
	in^2	ft^2			f_D	(ft)	(ft)	10%	ft
16	201.1	1.4	21.5	2.37E+06	2.13E-02	80.18	24.1	10.4	114.7
18	254.5	1.8	17.0	2.10E+06	2.07E-02	43.27	13.0	5.6	61.9
20	314.2	2.2	13.8	1.89E+06	2.02E-02	24.94	7.5	3.2	35.7
22	380.1	2.6	11.4	1.72E+06	1.98E-02	15.16	4.5	2.0	21.7
24	452.4	3.1	9.5	1.58E+06	1.94E-02	9.63	2.9	1.3	13.8
26	530.9	3.7	8.1	1.46E+06	1.91E-02	6.34	1.9	0.8	9.1
30	706.9	4.9	6.1	1.26E+06	1.85E-02	3.01	0.9	0.4	4.3
36	1017.9	7.1	4.2	1.05E+06	1.79E-02	1.17	0.4	0.2	1.7
40	1256.6	8.7	3.4	9.47E+05	1.76E-02	0.68	0.2	0.1	1.0
42	1385.4	9.6	3.1	9.02E+05	1.75E-02	0.53	0.2	0.1	0.8
44	1520.5	10.6	2.8	8.61E+05	1.73E-02	0.41	0.1	0.1	0.6
48	1809.6	12.6	2.4	7.89E+05	1.71E-02	0.26	0.1	0.0	0.4
54	2290.2	15.9	1.9	7.02E+05	1.68E-02	0.14	0.0	0.0	0.2
60	2827.4	19.6	1.5	6.31E+05	1.66E-02	0.08	0.0	0.0	0.1

Table 3. Suction pipeline velocities at Q, design.

No. of pumps =	2			Model DVC325c					
Q,pump =	6732.5	gpm		Design Q per pump					

Dia (in)	Area, A		V (ft/S)	Re	f	$h_{f,\ Major}$	$h_{f,\ Minor}$	S.F	$h_{f,\ Total}$
	in^2	ft^2			f_D	(ft)	(ft)	10%	ft
16	201.1	1.4	10.7	1.18E+06	2.14E-02	14.41	4.3	1.9	20.6
18	254.5	1.8	8.5	1.05E+06	2.09E-02	7.78	2.3	1.0	11.1
20	314.2	2.2	6.9	9.47E+05	2.04E-02	4.49	1.3	0.6	6.4
22	380.1	2.6	5.7	8.61E+05	2.00E-02	2.74	0.8	0.4	3.9
24	452.4	3.1	4.8	7.89E+05	1.97E-02	1.74	0.5	0.2	2.5
26	530.9	3.7	4.1	7.28E+05	1.94E-02	1.15	0.3	0.1	1.6
30	706.9	4.9	3.1	6.31E+05	1.89E-02	0.55	0.2	0.1	0.8

Table 4. NPSHa

h,atm =	33.9	ft	Atmospheric pressure.
h,vapor =	0.783	ft	Vapor pressure
h,lift_1 =	12.0	ft	Suction lift for pump placement at crest
h,lift_2 =	2.0	ft	Suction lift for pump placement at downstream
h,f_1 =	1.5	ft	Suction head loss for pump placement at crest
h,f_2 =	2.5	ft	Suction head loss for pump placement at downstream
S.F. =	3.39	ft	Safety Factor. 10% of atmospheric pressure
NPSHa_1 =	16.3	ft	NPSHa for pump placement at crest
NPSHa_2 =	25.3	ft	NPSHa for pump placement at downstream

Table 5. Static Head Required

HLOW Intake El =	528	ft	HLOW Intake El
Stage 1 Intake El =	450	ft	Stage 1 Intake El
LWL el =	455	ft	Low water level at coffer dam
HWL el =	465	ft	Coffer dam crest elevation
H_static, min,HLOW =	63	ft	Min static head required for HLOW Intake
H_static, max,HLOW =	73	ft	Max static head required for HLOW Intake
H_static,min,Stage 1 =	-15	ft	Min static head required for Stage 1 Intake
H_static,max,Stage 1 =	-5	ft	Max static head required for Stage 1 Intake

Table 6. Pump TDH Required

TDH,min_HLOW =	68.3	ft	Pump min TDH required for HLOW Intake
TDH,max_HLOW =	78.3	ft	Pump Max TDH required for HLOW Intake
TDH,min_Stage1 =	-9.7	ft	Pump Max TDH required for Stage 1 Intake. Gravity flow is feasible
TDH,max_Stage1 =	0.3	ft	Pump Max TDH required for Stage 1 Intake. Assumed gravity flow is feasible

DV 325c

Standard Sound Attenuated Pump
Fuel tank: 340 Gallon *1,287 liters*
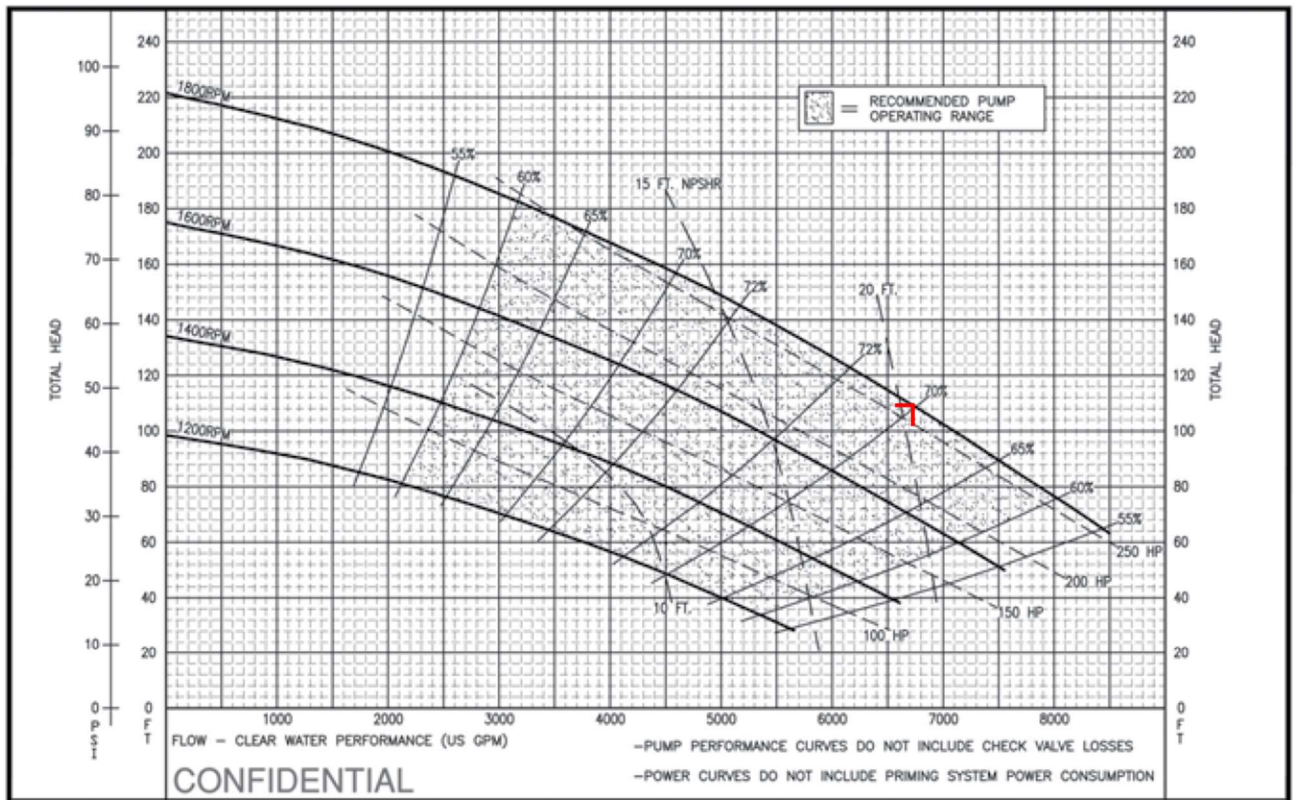
Fuel consumption: 12.5 GPH @ 1,800 RPM *47 liters per hour*

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Appendix D:

Diversion Extension Pipe Sizing

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Table of Contents

LIST OF TABLES

LIST OF FIGURES

D1. INTRODUCTION

The purpose of this appendix is to document our review of the available rainfall data, the hydraulic modeling results, and the design considerations that led to a recommended size for the diversion extension pipe. This appendix has been prepared to support the design of the cofferdam and diversion intake structure at the 90% level.

Bypass pumping would still be required during the Spring and Summer of Year 2 until the Stage 2 diversion system is constructed. Bypass pumping would also be required in Year 6 up to the high-level outlet works intake structure after the diversion system is shut down for completion of the low-level outlet works.

D2. RAINFALL DATA RECORDS

Streamflow data for the months of April to December at USGS gage 11169800 were used for the analysis. The following sections discuss the data and its processing for the hydraulic analysis.

D2.1   INFLOW TO ANDERSON RESERVOIR

The watershed area above Anderson Reservoir is approximately 193 square miles (mi^2). Approximately 120 mi^2 are regulated by Coyote Reservoir including approximately 109 mi^2 that is gaged (USGS, 2019). In order to estimate inflows to Anderson Reservoir it is necessary to estimate runoff from the 84 mi^2 of ungaged watershed.

The Initial Reservoir Dewatering Report (B&V and S&W, 2018) presented a method for adjustment of gaged streamflows to represent inflow to Coyote and Anderson reservoirs. The method relates drainage area and average annual precipitation, and resulted in the runoff ratios (with respect to the gage data) presented in Table D-1.

Table D-1. Adjustment of Gaged Streamflows to Represent Reservoir Inflow

WATERSHED	DRAINAGE AREA (MI^2)	MEAN ANNUAL PRECIPITATION (INCHES)	RUNOFF RATIO*
Coyote Creek near Gilroy (USGS gage 11169800)	109	27	--
Direct Inflow to Coyote Reservoir	120	27	1.1
Direct Inflow to Anderson Reservoir	75	24	0.6

* Calculated by method described in Initial Reservoir Dewatering Report (B&V and S&W, 2018)

D2.2   15-MINUTE STREAMFLOW DATA

15-minute streamflow data at USGS gage 11169800 was available for the period of 1-January, 2005, to 31-December, 2018. Dates of peak daily flow measurements in excess of

220-cfs[1] during the months of April through December were recorded. Events resulting in a peak daily flow of less than 220-cfs at the gage would be passable by a 6-foot diameter diversion extension pipe, and thus were not considered in the hydraulic modeling. The resulting streamflow events including peak 15-minute flow rate, peak daily flow rate, and ratio of peak 15-minute flow rate to peak daily flow rate are listed in Table D-2.

Table D-2. 15-minute Streamflow events at Coyote Creek near Gilroy (USGS, 2019)

DATE	PEAK FLOW (15-MINUTE [CFS])	PEAK FLOW (DAILY [CFS])	RATIO (15-MINUTE/DAILY)
April 3, 2006	2760	1760	1.6
April 12, 2010	898	545	1.6
April 13, 2012	1110	521	2.1
April 16, 2006	436	280	1.6
October 14, 2009	3090	497	6.2
December 2, 2012	3130	708	4.4
December 12, 2014	2100	845	2.5
December 13, 2009	485	324	1.5
December 16, 2016	435	287	1.5
December 19, 2010	1010	550	1.8
December 23, 2012	5820	1420	4.1
December 31, 2005	7440	2300	3.2
		Average	**2.7**

The percentage of peak daily flow was plotted against the percentage of time (see Figure D-1), showing a dimensionless representation of the streamflow event hydrographs. This enables visualization of the hydrograph shapes for consideration in the synthesizing of daily streamflow data records discussed in following section.

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[1] A storm event with peak daily flow of 220-cfs at USGS gage 11169800 routed through Coyote Reservoir and the cofferdam forebay results in a maximum flow of about 280 cfs through a 6-foot diameter pipe at maximum cofferdam forebay reservoir level El. 465 (two feet below the top of sheetpile) assuming that the Coyote Dam outlet works remains open during construction with enough water stored in Coyote Reservoir to maintain environmental release requirements. For comparison, a 30 cfs bypass pump system can pass an event with daily peak flow of about 110 cfs at USGS gage 11169800 assuming a starting cofferdam forebay reservoir level of 460 feet (7 feet below the top of sheetpile) and similar operation of Coyote Reservoir.
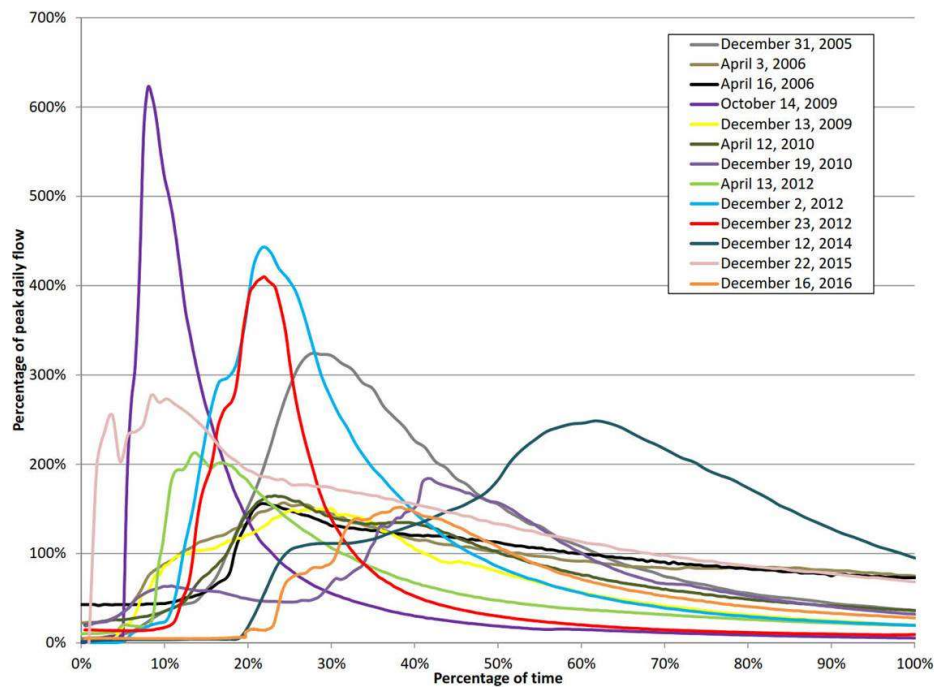
Figure D-1. Dimensionless Hydrographs from 15-minute streamflow records

D2.3   DAILY STREAMFLOW DATA

Daily average streamflow records at the USGS gage 11169800 were available for the periods 1-October, 1960 to 29-September, 1982, and 1-October, 2004 to 31-December, 2018. Dates with peak daily flow measurements in excess of 220-cfs during the months of April through December were recorded. These streamflow events and their peak daily flow rate values are listed in Table D-3.

Table D-3. Daily Streamflow Events at Coyote Creek near Gilroy (USGS, 2019)

DATE	PEAK FLOW (DAILY [CFS])
April 1, 1982	4400
April 2, 1974	884
April 7, 1963	1350
April 10, 1965	1270
April 15, 1963	547
April 21, 1963	487
April 22 1967	735
October 14, 1962	313
November 14, 1972	694
November 14, 1981	354
November 29, 1970	321

December 1, 1973	1030
December 2, 1970	650
December 6, 1966	1210
December 17, 1970	715
December 20, 1981	270
December 23, 1964	1190
December 25, 1979	344
December 27, 1973	992
December 29, 1965	792
December 30, 1981	282

The peak streamflow records listed in Table D-3 were synthesized into 15-minute data sets for the purpose of assessing how different sized diversion extension pipes would perform during these flow events. The method used to synthesize 15-minute data sets was to select a representative hydrograph from those presented on Figure D-1 and scale the daily streamflow events accordingly.

In order to be conservative in the analysis, one event with a higher peak flow ratio was selected for use as the dimensionless hydrograph template. Upon consideration of the larger storms, the December 23, 2012 storm was selected given that it was in the mid to upper ranges of the larger storms.

The majority of elevated streamflow (peak>220-cfs) occurred within a 48-hour period during the December 23, 2012 storm. This storm was therefore easily used to transform daily rain events with durations of 1-3 days by simply scaling the December 23, 2012 hydrograph either up or down according to the peak daily flow of the event in question.

Some events had longer elevated streamflow records, though it was generally observed that a distinct peak would still occur over some 1-3 day period during the event and the rising and falling limbs of the hydrographs would have relatively flat slopes. In these cases, the same hydrograph shape was used to emulate the period during which the peak flow was occurring, and the rising and falling limbs were simply assumed to have a constant flow as per the daily record. This allowed consideration of the antecedent flow conditions that would impact the available reservoir storage and the bypass systems ability to pass the events peaks during longer duration streamflow events.

D3. HYDRAULIC MODELING

A hydraulic model was created using the U.S. Army Corps of Engineering Hydraulic Engineering Center – River Analysis System (HEC-RAS) Version 5.0.5, released in June 2018. The model was used to simulate the routing of the rainfall events discussed in Section E-2 through Coyote and Anderson Reservoirs and through different sizes of diversion extension pipe. The following sections discuss the model and the modeling results.

D3.1   MODEL COMPONENTS

A simplified two-dimensional HEC-RAS model (see schematic in Figure D-2) was set-up to simulate hydraulic routing through Coyote and Anderson reservoirs. The following sections discuss the various components of the model.
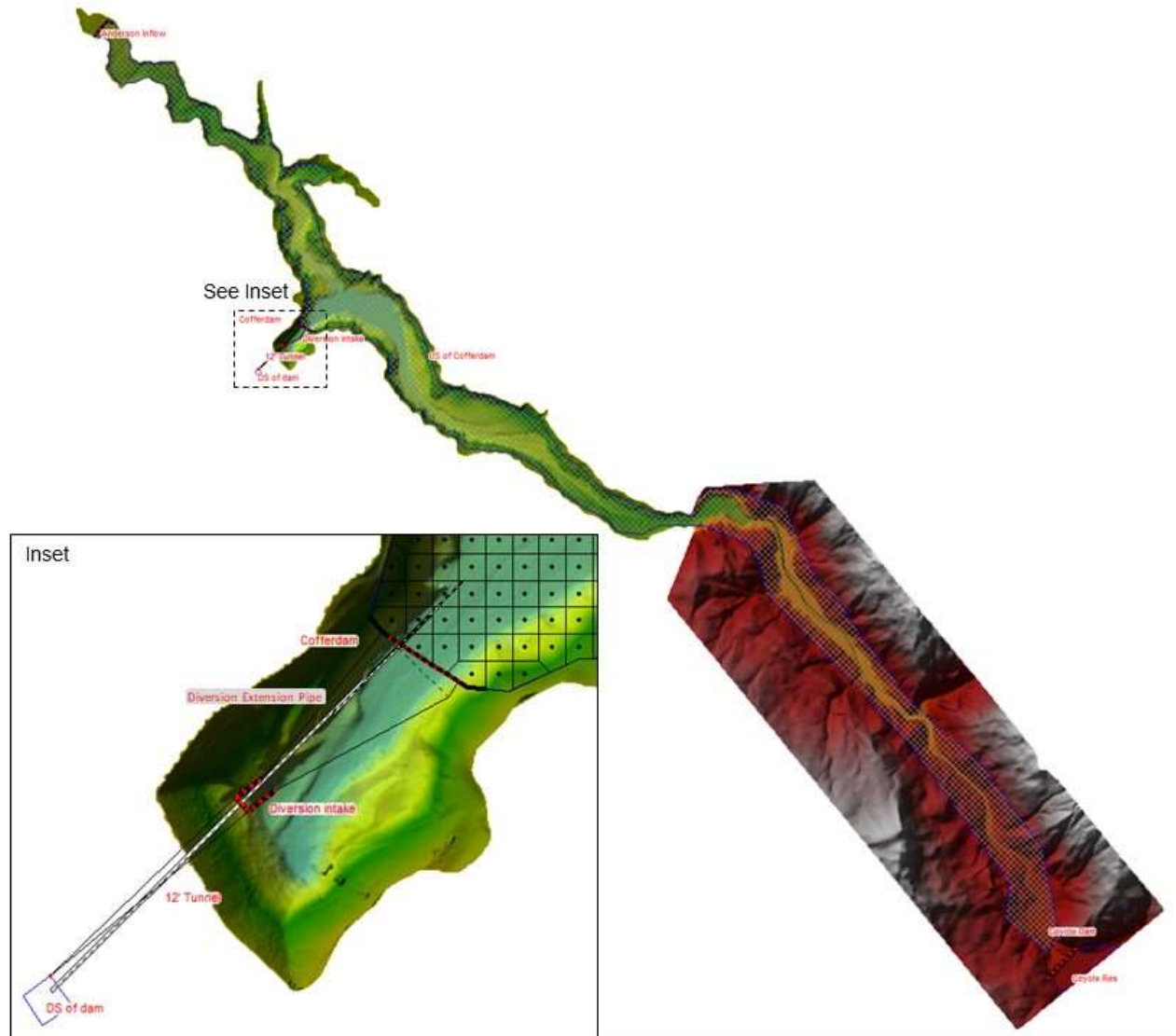
Figure D-2 Schematic of HEC-RAS model

D3.1.1   Boundary Conditions

The model assigned inflows into the system at two locations, one upstream of Coyote Reservoir and one along the northern boundary of Anderson Reservoir. Although this is a very simplified representation of the actual watershed, it was deemed to be a conservative approach given its under-estimating simplification of the time-of-concentration process.

D3.1.2   Storage-Elevation Relationships

Simulating the available storage volumes within the cofferdam forebay and Coyote Reservoir allowed the model to account for attenuation of peak flows in the two reservoirs. The storage-elevation relationship of cofferdam forebay was calculated using bathymetry data collected in 2016 (CLE, 2016). The storage-elevation relationships for the cofferdam forebay and Coyote Reservoir are presented on Figure D-3 and Figure D-4, respectively.
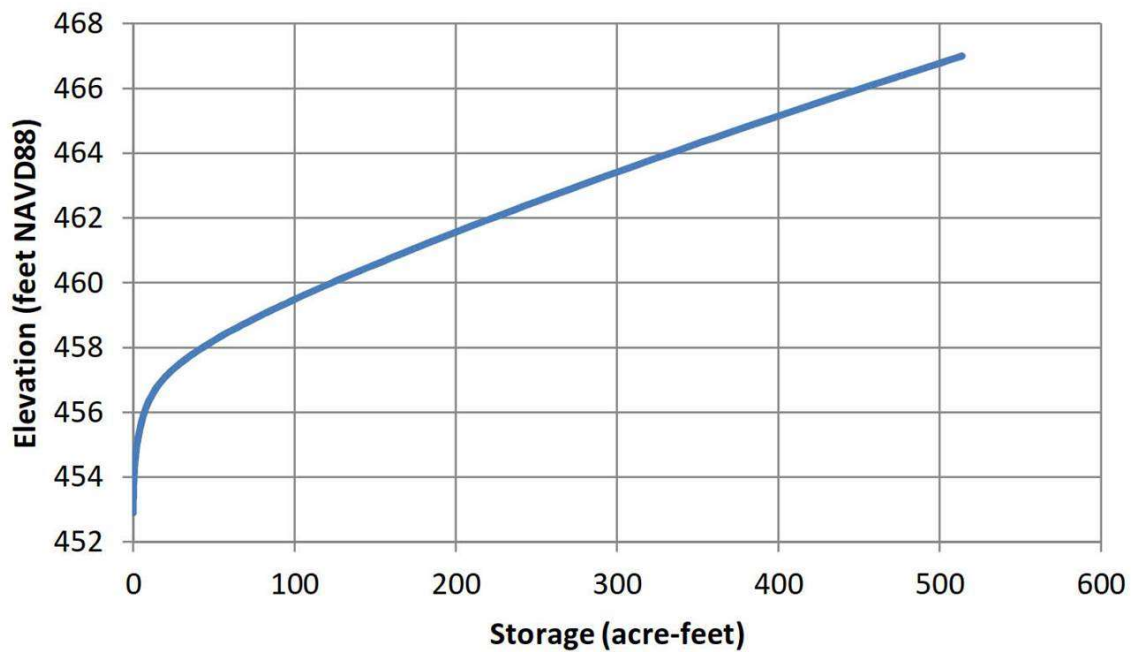
Figure D-3. Anderson Cofferdam Forebay Storage-Elevation Curve
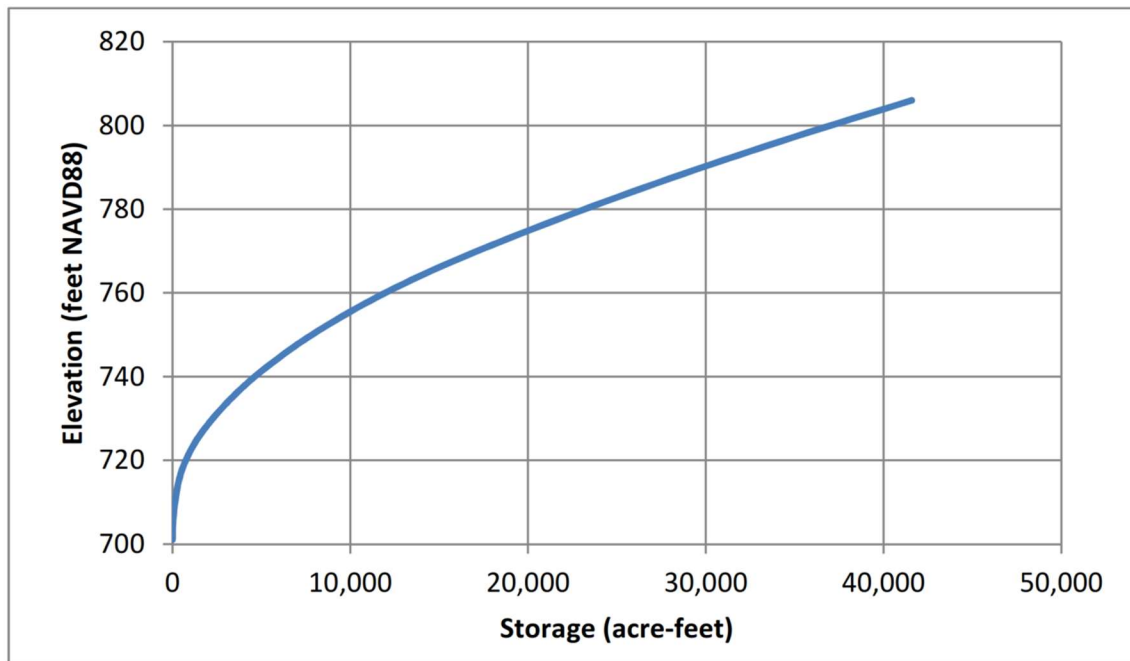
Figure D-4. Coyote Reservoir Storage-Elevation Curve (B&V, 2019)

D3.1.3   Stage Discharge Relationships

The stage discharge relationship for the 6-foot, 8-foot, and 10-foot diameter diversion extension pipes were generated in HEC-RAS using a 2D Flow Area Connection component. The diversion extension pipe was assumed to have a downstream invert of El. 450.5 and be 750-feet long. The top of pipe at the upstream end was set at El. 463; 2 feet below the crest of the cofferdam fill (El. 465). Grades for the 6-foot, 8-foot, and 10-foot pipes were 0.85%, 0.6%, and 0.33%, respectively. Pipe input parameters are summarized in Table D-4. The resulting stage discharge relationships for 6-, 8- and 10-foot diversion extension pipes, which were calculated within HEC-RAS, are presented on Figure D-5.

Table D-4. Diversion Pipe Input Parameters

PARAMETER	VALUE
Culvert Length (feet)	750
Upstream Top of Pipe Elevation (feet)	463.2
Downstream Invert Elevation (feet)	450.5
Pipe Slope	0.85% (6-foot), 0.6% (8-foot), 0.33% (10-foot)
Entrance Loss Coefficient	0.5
Exit Loss Coefficient	1.0
Manning's n	0.012
FHWA Chart #	1 – Concrete Pipe Culvert
FHWA Scale #	1 – Square edge entrance with headwall
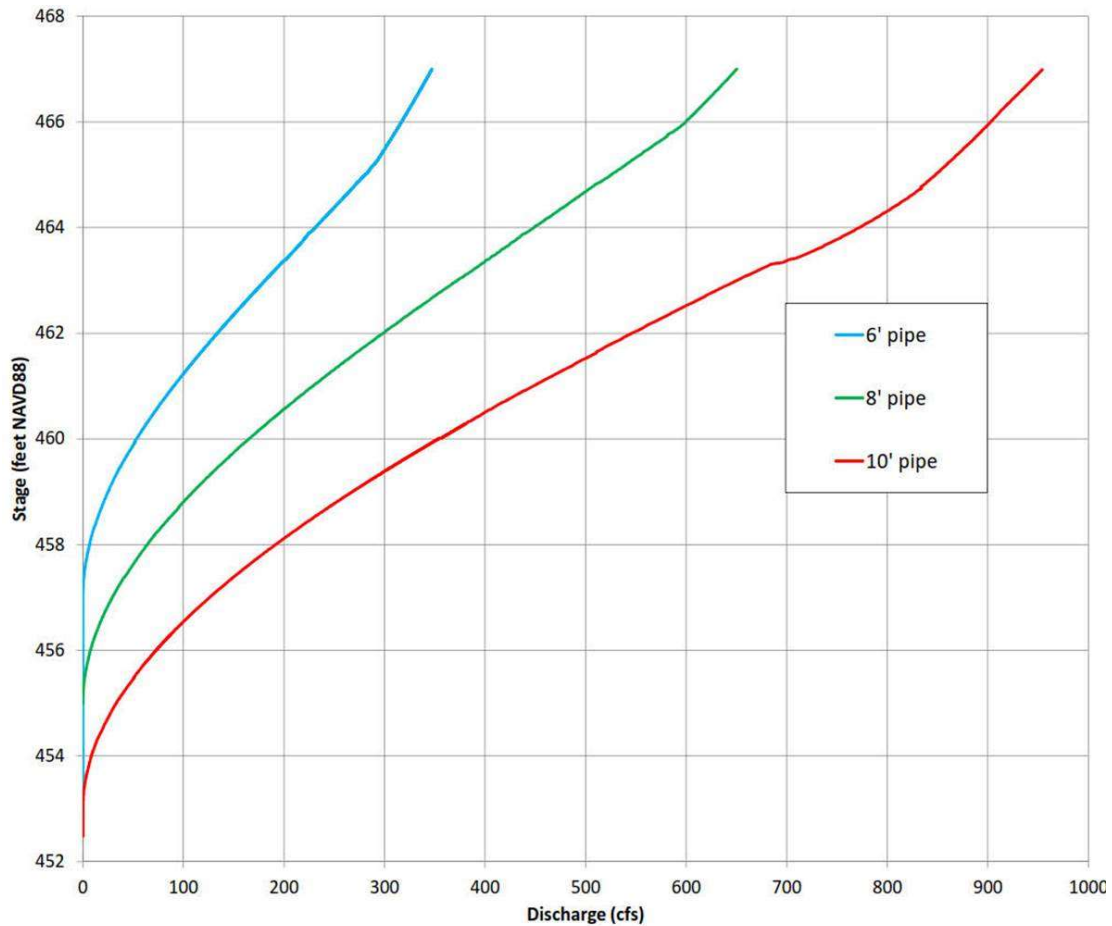
Figure D-5. Diversion Extension Pipe Rating Curves

The Coyote Dam outlet works were assumed to remain open during construction, assuming the volume of water stored in Coyote Reservoir was enough to maintain the environmental release requirements. This is further discussed in Section E3.1.5. The elevation-discharge relationship for Coyote Dam is presented on Figure D-6.
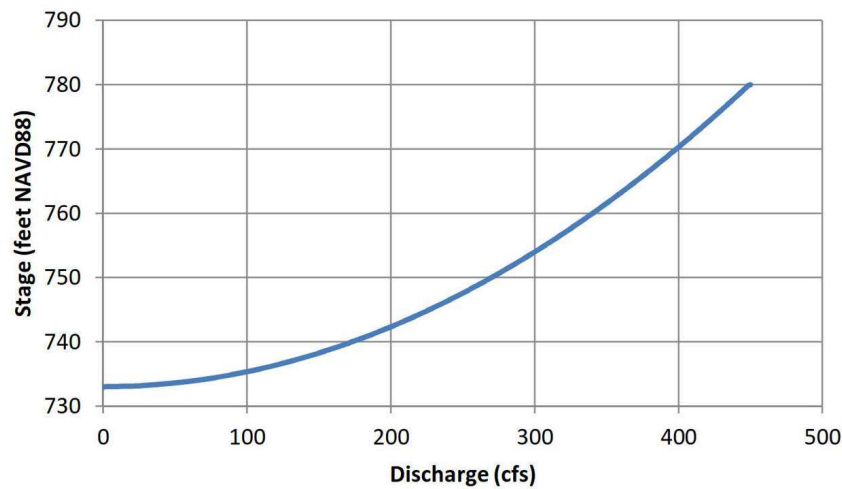
Figure D-6. Coyote Dam Rating Curve (B&V, 2019)

D3.1.4   Initial Conditions

The initial water level in the cofferdam forebay was assumed to be at the upstream diversion pipe extension invert (El. 457 for 6-foot pipe, El. 455 for 8-foot pipe, and El. 453 for 10-foot pipe). This was the level that the forebay would drain to given sufficient time since the last streamflow event. If records showed streamflow in the days preceding an event, this streamflow was included in the analysis to allow appropriate reservoir water levels when the event peaks arrived.

The initial water level in Coyote Reservoir was set according to the rule that enough water must be retained to provide a minimum 5-cfs environmental release for the rest of that year. The corresponding minimum water surface elevations for each month were assigned as the initial water level in Coyote Reservoir as shown on Table D-5.

Table D-5. Minimum Stage and Storage Requirements for Coyote Reservoir

MONTH	MINIMUM STORAGE VOLUME (ACRE-FEET)	MINIMUM WATER SURFACE ELEVATION (FEET)
March	8,300	751.3
April	7,749	749.8
May	7,114	748
October	4,046	737.9
November	3,653	736.3
December	3,297	734.8

Values obtained from synthesized data

D3.1.5   Results

Table D-6 presents a list of streamflow events that were considered in the modeling and the results of modeling indicating the smallest diversion extension pipe size (6, 8 or 10 feet) that can bypass the flow event or overtopping.

Table D-6. Hydraulic Routing Results

DATE	PEAK FLOW (15-MINUTE [CFS])	PEAK FLOW (DAILY [CFS])	RESULT
April 1, 1982	18,034*	4,400	Cofferdam overtopped
April 2, 1974	3,623*	884	Cofferdam overtopped
April 3, 2006	2,760	1,760	Cofferdam overtopped
April 7, 1963	5,533*	1,350	Cofferdam overtopped
April 10, 1965	5,205*	1,270	Cofferdam overtopped
April 12, 2010	898	545	8' pipe passes flow
April 13, 2012	1,110	521	8' pipe passes flow
April 15, 1963	2,242*	547	10' pipe passes flow
April 16, 2006	436	280	8' pipe passes flow
April 21, 1963	1,996*	487	8' pipe passes flow
April 22 1967	3,012*	735	Cofferdam overtopped
October 14, 1962	1,283*	313	6' pipe passes flow
October 14, 2009	3,090	497	Cofferdam overtopped
November 14, 1972	2,844*	694	Cofferdam overtopped
November 14, 1981	1,451*	354	8' pipe passes flow
November 29, 1970	1,316*	321	8' pipe passes flow
December 1, 1973	4,222*	1,030	Cofferdam overtopped
December 2, 1970	2,664*	650	Cofferdam overtopped
December 2, 2012	3,130	708	10' pipe passes flow
December 6, 1966	4,959*	1,210	Cofferdam overtopped
December 12, 2014	2,100	845	10' pipe passes flow
December 13, 2009	485	324	6' pipe passes flow
December 16, 2016	435	287	6' pipe passes flow
December 17, 1970	2,930*	715	Cofferdam overtopped
December 19, 2010	1,010	550	8' pipe passes flow
December 20, 1981	1,107*	270	6' pipe passes flow
December 23, 1964	4,877*	1,190	Cofferdam overtopped
December 23, 2012	5,820	1,420	Cofferdam overtopped
December 25, 1979	1,410*	344	6' pipe passes flow
December 27, 1973	4,066*	992	Cofferdam overtopped
December 29, 1965	3,246*	792	Cofferdam overtopped
December 30, 1981	1,156*	282	6' pipe passes flow
December 31, 2005	7,440	2,300	Cofferdam overtopped

Values obtained from synthesized data

Table D-7 summarizes Table D-6 into the total number of storms during the first and last halves of each month, the corresponding number of storms that are bypassed for each pipe size, and the number of storms that would overtop the cofferdam sheetpile (El. 467).

Table D-7. Hydraulic Routing Results Summary for Storms Greater Than 220 cfs

DESCRIPTION	APR 1-15	APR 16-30	OCT 1-15	OCT 16-31	NOV 1-15	NOV 16-30	DEC 1-15	DEC 16-31	TOTAL
Number of Storms	8	3	2	0	2	1	6	11	33
Bypass with 6' Pipe	0	0	1	0	0	0	1	4	6
Bypass with 8' Pipe	2	2	1	0	1	1	1	5	13
Bypass with 10' Pipe	3	2	1	0	1	1	3	5	16
Cofferdam overtopped	5	1	1	0	1	0	3	6	17

As shown in Table D-7, it is estimated that 6-foot, 8-foot, and 10-foot diversion extension pipes would bypass:

- April 1 to December 31 - 18%, 39%, and 48% of historic storms greater than 220 cfs, respectively
- April 15 to December 15 - 14%, 43%, and 57% of historic storms greater than 220 cfs, respectively
- April 15 to November 30 - 13%, 62%, and 62% of historic storms greater than 220 cfs, respectively

The number of historic storms that would pass through the diversion extension pipes that would not have been passed by the 30 cfs bypass pump was estimated by counting up the number events recorded at USGS gage 11169800 that had peak daily flows greater than 110 cfs and less than 220 cfs. The resulting number of storms was 14 with 8 occurring in April, 2 occurring in November, and 4 occurring in December. In addition to the storms from Table D-7, Table D-8 also incorporates the storms between 110 cfs and 220 cfs to capture the estimated benefit of the diversion extension pipe over the 30 cfs bypass pump system.

Table D-8. Hydraulic Routing Results Summary for Storms Greater Than 110 cfs

DESCRIPTION	APR 1-15	APR 16-30	OCT 1-15	OCT 16-31	NOV 1-15	NOV 16-30	DEC 1-15	DEC 16-31	TOTAL
Number of Storms	15	4	2	0	2	3	7	14	47
Bypass with 6' Pipe	7	1	1	0	0	2	2	7	20
Bypass with 8' Pipe	9	3	1	0	1	3	2	8	27
Bypass with 10' Pipe	10	3	1	0	1	3	4	8	30
Cofferdam overtopped	5	1	1	0	1	0	3	6	17

As shown in Table D-8, it is estimated that 6-foot, 8-foot, and 10-foot diversion extension pipes would bypass:

1. April 1 to December 31 - 43%, 57%, and 64% of historic storms greater than 110 cfs, respectively
2. April 16 to December 15 - 33%, 56%, and 67% of historic storms greater than 110 cfs, respectively
3. April 16 to November 30 - 36%, 73%, and 73% of historic storms greater than 110 cfs, respectively

D4. DIVERSION EXTENSION PIPE DESIGN CONSIDERATIONS

Extension of the diversion pipe requires trenching, pipe laying and welding, and trench backfilling for approximately 750 feet from the diversion intake structure to the upstream side of the cofferdam. The pipe would be trenched through the right abutment for the cofferdam. At the cofferdam, the pipe would likely require a reinforced concrete encasement to protect the pipe from large loads due to off-highway dump trucks crossing the cofferdam that will be hauling embankment materials to and from stockpile areas SA-C and SA-H and the Packwood Gravel Borrow Pit.

Connection to the diversion intake structure will involve a straight forward modification to the wall of the intake structure that would be incorporated in the 90% diversion intake structure design. The modification would be similar for each of the pipe sizes being considered.

The estimated direct construction cost of the pipe including installation is provided in Table D-9. The table includes estimated costs for both steel and reinforced concrete pipes, which are presently both considered suitable alternatives.

Table D-9. Costs of Pipes (Including Installation) in 1st Quarter 2019 Dollars

DESCRIPTION	STEEL PIPE	REINFORCED CONCRETE PIPE
6' Pipe	$540,000	$340,000
8' Pipe	$765,000	$820,000
10' Pipe	$1,200,000	$1,225,000

D5. CONCLUSIONS AND RECOMMENDATIONS

The routing results demonstrate that a diversion extension pipe substantially reduces the potential for overtopping of the cofferdam in April, November, and December and thereby increases the number of days that would be available during the shoulder seasons for work in the reservoir area. Based on available historic flow data, a 10-foot diversion extension pipe will bypass 30 of 47 events that would not be able to be bypassed using a 30 cfs pumping system. Extension pipes of 6-foot and 8-foot would bypass 20 and 27 of the 47 events, respectively.

It is recommended that moving forward into 90% design that a diversion extension pipe be incorporated into the project. It is recommended that the bypass pipe have a 10-foot diameter to maximize the reduction of risk of cofferdam overtopping.

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Appendix E:

Excerpts from Selected References Related to Displacement Method of Construction of Embankments on Soft Soils

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Lambrechts & Kinner (1988)

The Great Salt Lake Causeway—A Calculated Risk Revisited

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SYNOPSIS: The construction of the Great Salt Lake Causeway involved several calculated risks. Original design assumptions on lake level and consolidation settlement were not realized, creating a unique situation where the critical time for stability of this embankment was not necessarily at the end-of-construction. Along more than half of the Causeway's 12-1/2 mile length, consolidation and strength gain has apparently been inhibited by a layer of salt. Because it was anticipated that calculated Factors of Safety for current conditions would be close to the 1.0 originally used, a comparative approach to stability evaluations was adopted. In this approach, Factors of Safety calculated for known, past stable conditions were compared with those predicted for future conditions. Judgements of future Causeway stability were made by comparing Factors of Safety with time. The presence of a salt layer in the foundation of a portion of the Causeway's length renders exact solution of stability intractable to usual analytical procedures.

INTRODUCTION

In his 1964 Terzaghi Lecture, Casagrande (1965) described the design and construction of this 12-1/2 mile long embankment across the deepest portion of the Great Salt Lake, location indicated in Figure 1, as an outstanding example of a calculated risk. Completion of the Causeway in 1959 required the application of considerable engineering judgement as the original design of the embankment underwent several empirical modifications based on the results of test fills and on actual construction failures. Initial "assumed" risks included;

1. Use of a design Factor of Safety close to 1.0, for expected greater economy of a less conservative design, but accepting the increased risk of failures.

2. Selection of crest elevation 4212[1], based on lake levels over the previous 30 years and the anticipation that there was a general downward trend to the level of this terminal lake, see Figure 2. This crest elevation was more than 6 ft. lower than the rails on the 55 year old timber trestle that it was to replace.

3. The expectation that consolidation of the soft foundation clays would be on the order of 4 to 8 ft., and that this would occur within several years of construction, thus yielding a steady increase in stability.

However, the level of the Great Salt Lake, a lake with no natural outlet, has fluctuated nearly 20 ft. since 1959, see Figure 2. Recent historically high lake levels and continuing Causeway settlement due to foundation clay consolidation, also illustrated in Figure 2, have forced Southern
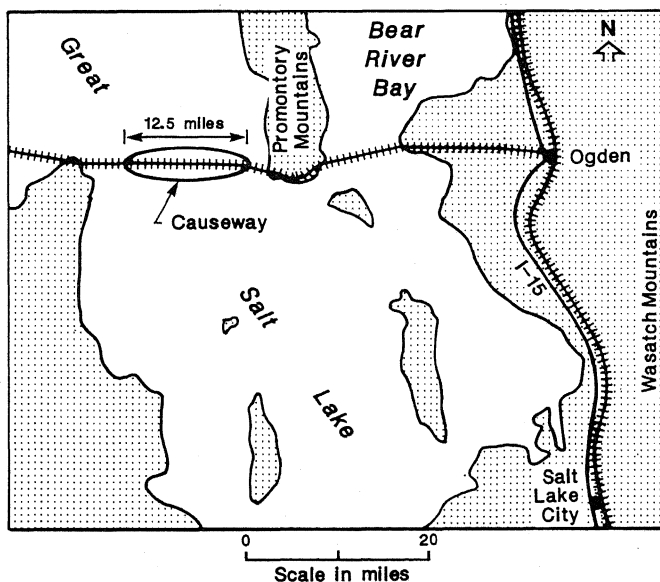
Figure 1. Location of the Great Salt Lake Causeway.

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[1] All Elevations refer to Southern Pacific's Hood's Datum, and are therefore 3.4 ft. above elevations based on USGS datum.
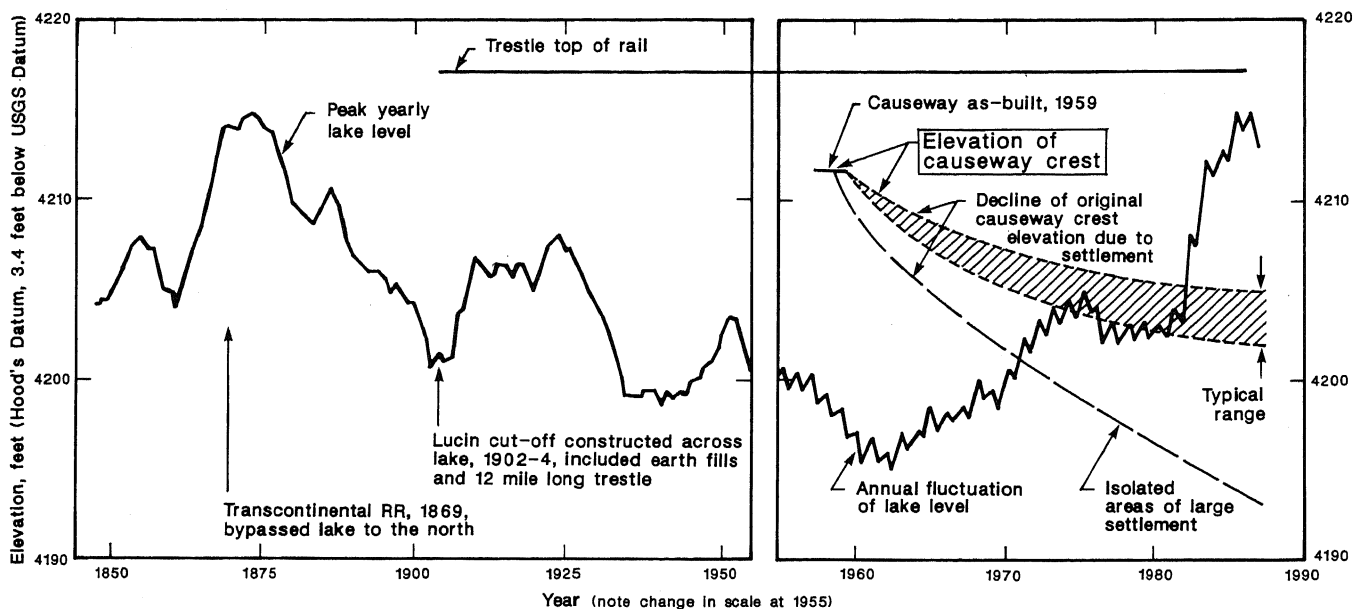
Figure 2. Levels of the Great Salt Lake and Causeway Crest.

Pacific railroad to add significant amounts of fill to the crest to maintain adequate freeboard. Filling has increased stresses on the foundation clays and necessitated re-assessment of Causeway stability. Studies have shown that a unique situation has developed wherein the end-of-construction condition was not necessarily the critical time for stability.

A comparative approach to stability assessments was adopted in which the Factors of Safety were calculated at selected cross-sections for differing conditions that existed at various times since construction. Changes in Factor of Safety from those calculated for past, stable conditions were then evaluated to assess present and future stability.

Although stability was found to have improved where the Causeway is directly founded on a clay foundation, the presence of a brittle salt layer beneath much of the Causeway's length renders stability calculation by "usual" procedures intractable. Assessment of stability in these areas still requires engineering judgement, thus continuing the calculated risks.

SUBSURFACE CONDITIONS ALONG THE CAUSEWAY

Although the Great Salt Lake basin has in places over 7,000 feet of sediment, only those strata within 150 to 200 feet of the present mudline were of consequence to Causeway stability. Below that level, stiff dessicated clay from an evaporative lake cycle is present. The overlying sediments are predominantly soft, plastic organic clays that exhibit brittle behavior in compression tests,

Casagrande (1959). Fine sand partings are found in deeper strata of the soft clays.

Of particular importance to Causeway stability is a stratum of Glauber's Salt. It is present in the deeper lake areas and is buried under about 25 ft. of the very soft sediments. This hydrated sodium sulfate was deposited during the evaporative aftermath of Lake Bonneville. The stratum contains a wide variety of salt compositions, each separated from the next by clay seams, as described by Eardley (1962). The Glauber's Salt increases in thickness from west to the east, being less than 1 ft. thick along the western 2-1/2 miles of the Causeway, up to 20 ft. beneath the central 6 miles, and as much as 45 ft. thick near the east side.

To evaluate soil shear strengths required for the recent stability studies, an extensive program of soil sampling and laboratory testing was undertaken in 1984. Borings were made at five different locations both through the centerline of the embankment and over-water through the counter-weight berms. Because SHANSEP procedures (Ladd and Foott, 1974) were used to assess clay shear strength, over 100 consolidation tests were performed to determine profiles of maximum past pressure.

The stress profiles shown in Figure 3 indicate that far less consolidation has occurred in clay overlain by the salt. This was an important discovery, because shear strength increase would be similarly less than where the salt is present. The Glauber's Salt stratum has apparently inhibited drainage from the underlying foundation clays. Thus less than 35 percent of the eventual consolidation has occurred in most salt areas.

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Embankment on clay foundation, no salt layer

Embankment on thin salt
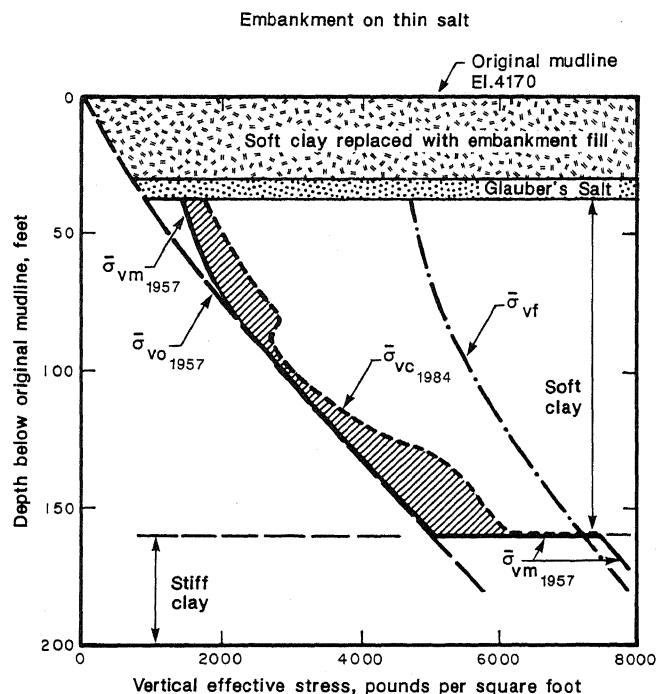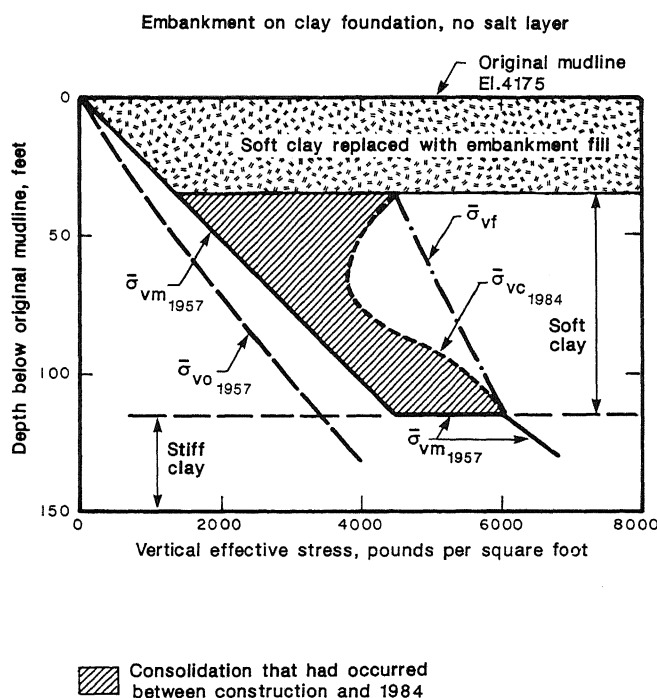
Figure 3. Stress Profiles Indicating Consolidation Since Causeway Construction.

where: $\overline{\sigma}_{vo\ 1957}$ = preconstruction vertical effective stress

$\overline{\sigma}_{vm\ 1957}$ = preconstruction maximum past pressure based on consolidation tests

$\overline{\sigma}_{vc\ 1984}$ = vertical effective stress in 1984 (equals $\overline{\sigma}_{vm\ 1984}$ where $\overline{\sigma}_{vc\ 1984}$ is greater than $\overline{\sigma}_{vm\ 1957}$)

$\overline{\sigma}_{vf}$ = final vertical effective stress under Causeway centerline at full consolidation

ORIGINAL CONSTRUCTION

A plan of the completed Causeway circa 1959, a profile showing fill and salt thickness, and two typical cross-sections are shown in Figure 4.

Casagrande (1959 and 1965) and Newby (1980) describe the design and construction. In brief, the embankment was generally constructed to a 60 ft. wide crest that was 12 ft. above the average lake level. Side slopes of the earth and rock fill embankment were 2 horiz. to 1 vert. A key design element was the removal by dredging of 20 to 25 ft. of the softest lake bottom sediments from beneath the main body of the fill. Along much of its length, the main fill was placed on the

Glauber's Salt. Counter-weight berms were placed adjacent to most sections of the main fill. The depth and width of the dredged trench, and berm width and locations are shown in Figure 4.

During construction, test fills were constructed in each of the different foundation areas to provide insight on performance and to verify the design sections used because the design Factor of Safety was close to 1.0. Unexpected construction failures that occurred emphasized the fact that as-constructed stability was marginal. These failures resulted in several design revisions which became largely empirical, Casagrande (1965).
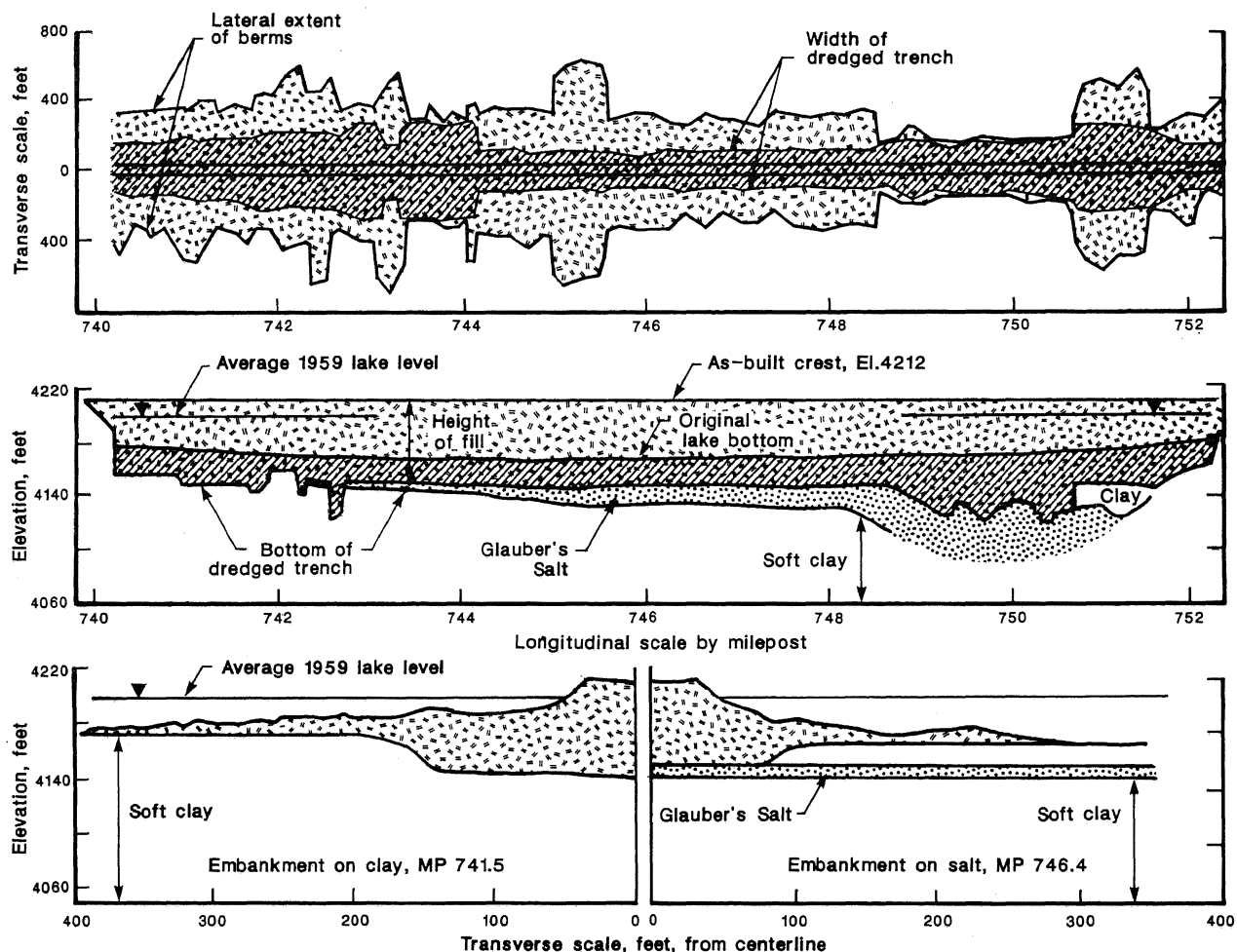
Figure 4. Plan, Profile, and Typical Cross-Sections of Causeway.

COMPARATIVE APPROACH TO STABILITY ASSESSMENTS

Unlike most embankments on soft ground, the critical time for Causeway stability was not necessarily at the end of construction. Falling and rising lake levels and the 12 to 25 ft. of fill added to the crest to accommodate settlement and rising lake level have caused changes in effective stresses on the foundation clays. Shear strength of the foundation clays has been very slow to increase in some areas.

In light of the above risks, a comparative approach was adopted for assessment of current and future stability. The Factors of Safety at the end of construction and other selected times since construction were calculated and compared. Judgements concerning stability were then made by comparing Factors of Safety for current and future conditions with those for past times when stable conditions were known to exist.

A few sections representative of the various constructed geometries along the 12-1/2 mile long Causeway were selected for detailed stability analysis. Stability analyses were made primarily for the West Side areas where the Causeway is solely on a clay foundation. These results and limited other analyses were used to temper judgements and assessments of instability risk where the the stiff salt layer is present.

Stability analyses were made for the varying Causeway geometries, lake levels and shear strength profiles that were applicable to the past and present conditions at each section. Several possible future conditions were also analyzed to provide insight for the assessment of future risks of instability. The major changing conditions that were evaluated are listed in Table I.

The geometry of the Causeway surface was taken from cross-sections made at the end of

TABLE I. Changing Conditions Evaluated in Comparative Stability Analyses

YEAR	LAKE ELEVATION	ACCUMULATED SETTLEMENT	BASIS FOR CLAY STRENGTH (feet)	CAUSEWAY CONDITIONS/CHANGES
1959	4200	0	$\bar{\sigma}_{vm_{1957}}$	END OF CONSTRUCTION
1963	4195	2	$\bar{\sigma}_{vm_{1957}}$	MINOR SETTLEMENT
1969	4198	4	$\bar{\sigma}_{vm_{1969}}$ (INTERPOLATED)	FILL ADDED TO RAISE SUBGRADE BACK TO AS-BUILT (EL.4212), SOME BERM EROSION
1984	4210	7	$\bar{\sigma}_{vm_{1984}}$	FILL ADDED TO RAISE SUBGRADE TO EL.4214, BERM EROSION / ACCRETION
FUTURE	VARIED 4213 TO 4190	7	$\bar{\sigma}_{vm_{1984}}$	FILL ADDED TO RAISE SUBGRADE TO EL.4217

construction, and in 1966 and 1984. Settlement was distributed through the cross-section by assuming; 1. full settlement under the main body of the embankment, 2. no settlement under the berms, 3. linear variation in between. For perspective on the impact of "other factors" on instability risk, analyses were made for possible future lower and higher lake levels, addition or removal of fill from the crest, and berm thickness changes (field studies have indicated that erosion and accretion may have occurred).

Stability analyses were made using the Modified Bishop method for circular surfaces and Janbu method for non-circular surfaces, as available in the computer program STABLE, Boutrup (1977). A limited number of Morgenstern-Price analyses (ICES-LEASE) were performed which indicated Janbu to be approximately 10 percent conservative.

The shear strength of the foundation clays, both with depth and laterally from the main body of the fill, were determined by SHANSEP procedures for end-of-construction and later times. A stress ratio, $s_u/\bar{\sigma}_{vm}$, of 0.225 was established on the basis of laboratory undrained triaxial compression and extension tests and direct simple shear tests. The in-situ strength ratio was also estimated by back-analysis of two construction failures, which indicated somewhat lower values. A strength ratio of 0.20 was finally selected.

STABILITY OF EMBANKMENT ON CLAY

The section at MP 741.5 was evaluated because it is typical of "normal" West Side conditions where there is only clay in the foundation strata and did not experience a construction failure. The Factors of Safety calculated for this cross-section are summarized in Table II, as are the results of limited analyses for a cross-section at MP 747.6 where the salt layer is present.

The Factors of Safety calculated for conditions at the end of construction and four years later, when the Great Salt Lake was about 6 ft. lower, are essentially the same, and just slightly above unity. This agrees with design reports that the original design Factor of Safety was close to 1.0. The 1963 Factor of Safety was perhaps slightly higher than that calculated because some slight clay strength increase likely occurred, but was not considered in these analyses.

TABLE II. Calculated Changes in Factor of Safety with Time

YEAR	CALCULATED MINIMUM FACTOR OF SAFETY JANBU (NONCIRCULAR)	
	NON-SALT (MP 741.5)	SALT FOUNDATION (3) (MP 747.7)
1959	1.06	1.37
1963	1.04	(4)
1969	1.24	(4)
1984	1.25	(4)
FUTURE (1)	1.15	1.33
FUTURE (2)	1.25	1.37

1. SAME LAKE LEVEL AS 1984 CONDITIONS, CREST EL.4217.
2. LAKE LEVEL 3 FEET ABOVE 1984, CREST EL.4217.
3. SALT LAYER THICKNESS = 12 FEET, ASSUMED SALT SHEAR STRENGTH = 3600 psf.
4. NOT CALCULATED

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The Factor of Safety calculated for 1969 conditions was 20 percent greater than original, a substantial improvement. This increased stability was primarily a consequence of increased shear strength in the foundation clay which more than offset the affect of the fill that was added to the crest in 1969 to compensate for 4 ft. of settlement that had occurred. The 1969 lake level was 2 ft. below 1959 level.

Calculations for 1984 conditions showed that the cumulative effects of continued foundation clay strength increase and lake level 10 ft. above the as-built level counteracted the destabilizing effects of the weight of an additional 6 to 7 ft. of fill placed on the crest in 1984 and the apparent erosion of about 2 ft. of material from the berms. The Factor of Safety was about the same as in 1969, about 20 percent greater than the as-built condition.

The effect of future changes in lake level, elevation of the crest, and erosion of the berms on Factor of Safety for Sta. 3550 were evaluated in a series of parameter studies. The results are shown on Figure 5 as change in Factor of Safety for each variable alone, the others being held constant at the 1984 conditions.
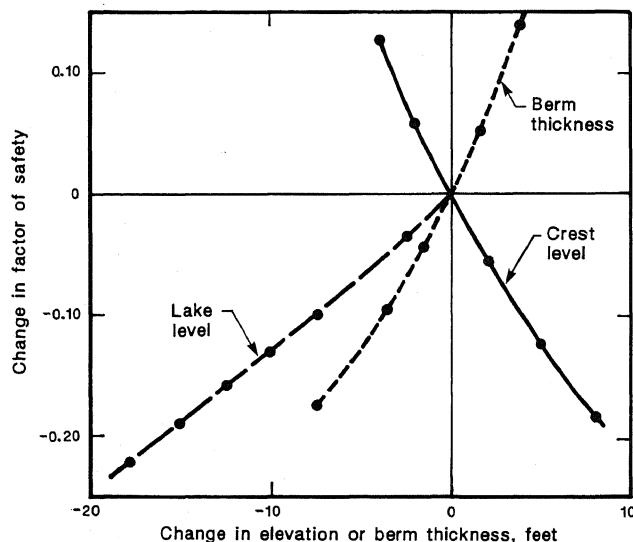
Figure 5. Effect of Various Parameters on Calculated Factor of Safety.

Changes in lake level were found to have about half of the effect on Factor of Safety as changes in either crest elevation or berm thickness. The direction of each effect is obvious. The Factors of Safety for two of the future conditions are included in Table II.

Based on these results, it was concluded that where there is only soft clay in the foundation, present and near-term, future Causeway stability would be greater than the as-built conditions. However, if lake levels recede below Elev. 4205, 5 ft. above the 1959 level, the calculated Factor of Safety would decrease to near the marginally stable values calculated for 1959 conditions.

PROBLEMS OF SALT OVER SOFT CLAY FOUNDATION

Stability calculations indicated that a working shear strength of the salt layer of between 10 and 25 times that of the clay at the same level was necessary for stability, i.e. Factor of Safety greater than 1.0. But it was not considered possible to make a meaningful analysis of Factor of Safety for the Causeway where the fill is on stiff salt over the soft foundation clays for the following reasons:

1. The salt is extremely heterogeneous, the spacing and frequency of clay seams varies with elevation and location.

2. The overall behavior of the salt is not understood, but is probably not adequately represented by laboratory compression tests on core samples.

3. The salt has probably experienced bending stresses due to differences in settlement between zones under the main body of the fill and beneath the lightly loaded berms, and compression due to the weight of the fill, but the effect of these changes on stratum strength is unknown.

4. There is strain incompatibility between the stiff, brittle salt and the soft clays below, although salt can typically accommodate large creep strains.

Limited comparative stability analyses were performed for insight on the magnitude of changes in Factor of Safety. The results of some of these analyses, presented in Table II, indicate little change in stability from as-built conditions. This is due primarily to the small gains in foundation clay shear strength. Assessments of present and future stability were therefore based substantially on judgement. Raising the crest to maintain freeboard was still considered a calculated risk that was necessarily taken to continue rail traffic over the Causeway.

Caution and continual monitoring of embankment performance were recommended, and contingency plans for adding fill to the berms were developed in case settlement rates became excessive.

In two areas, each about 1/2 mile long, settlement rates have recently been on the order of 1 to 1-1/2 ft. per year which is 2 to 4 times the "normal". Both areas have been identified as probably having a locally weaker

salt stratum. Inclinometers recently installed offshore have shown there to be significant, ongoing, lateral displacements of soft clay below the salt, nearly 30 years after construction. The question of the future performance of these areas and the possible development of other similar areas remains a calculated risk.

CONTINUING CALCULATED RISKS

Today, many uncertainties and limitations still exist which make assessments of Causeway stability, a continued calculated risk. Principal contributing factors are:

1.   Likely variations in foundation strata conditions from those assumed based on the limited number of borings made along the 12-1/2 mile long embankment.

2.   The presence of the Glauber's Salt, a brittle yet ductile material, that is heterogeneous with depth and lateral extent. Its strength may change with time due to deformation from consolidation of the underlying clays.

3.   Very slow consolidation and strength gain in the clays below the Glauber's Salt.

4.   Lack of precision in the Factor of Safety calculation due to inaccuracy in determining soil parameters and soil and fill stratification.

5.   Inability to adequately accommodate the salt layer in current stability analyses due to its strain incompatibility with the fill and foundation clays.

6.   Inability to analyze more than a few representative cross-sections along the 12-1/2 mile long embankment due to cost and time constraints.

CONCLUSIONS

Original design expectations on lake levels and consolidation of the foundation clays below the salt stratum have not come to fruition. Consequently, the recent rise of the Great Salt Lake to historic levels and the need to add fill to the Causeway crest have created a unique situation wherein the end-of-construction was not necessarily the critical time for stability.

It was anticipated that Factors of Safety would again be close to the 1.0 adopted in design. The comparative approach to stability assessments, adopted for evaluations of current conditions and recent elevated lake levels, indicated 10 to 20 percent greater stability than at the end-of-construction where the Causeway is founded on soft clay.

However, embankment stability remains an intractable problem for more than half its length where the Causeway is founded on interbedded salt above soft clays, due to problems of salt/clay strain incompatibility and salt stratum heterogeneity and probable changes since construction. Therefore, the results of analyses made for the no-salt West Side were used for insight in stability considerations. However, the substantial amount of engineering judgement required in assessing stability continue to make such evaluations calculated risks.

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DREDGED MATERIAL RESEARCH PROGRAM
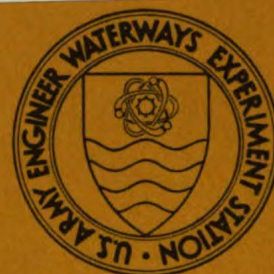
DESIGN AND CONSTRUCTION OF RETAINING DIKES FOR CONTAINMENT OF DREDGED MATERIAL

by

David P. Hammer and Edward D. Blackburn

U. S. Army Engineer District, Savannah
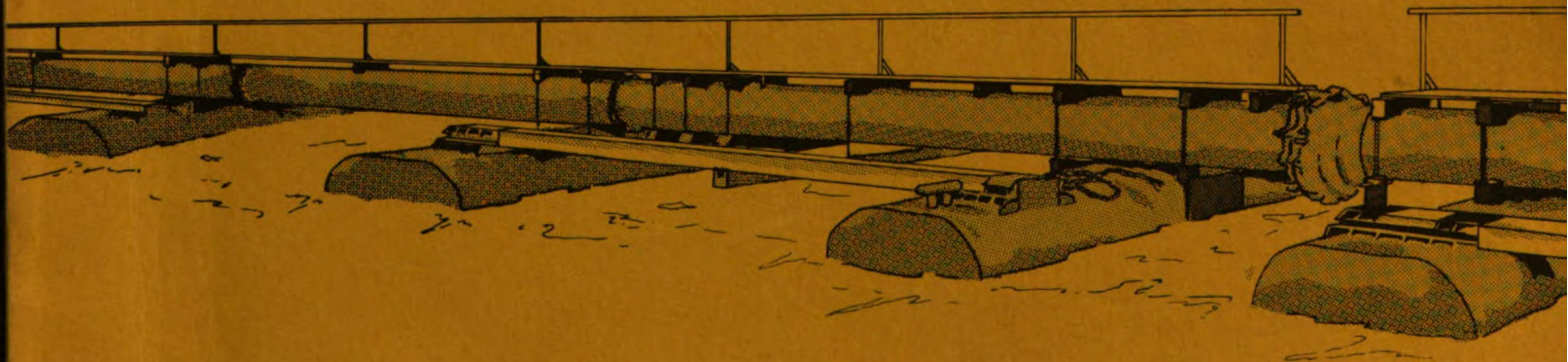Soils Section
P. O. Box 889, Savannah, Georgia 31402

instance, it is impractical, if not impossible, to construct a steep-sloped compacted dike on a soft foundation. Conversely, it is usually unnecessary to specify a compacted dike where a soft foundation dictates a section with flat slopes; rather, it would be more reasonable to specify a method of construction which, by its very use, results in flatter slopes such as traffic compaction or hydraulic fill. The important thing is to make all of the variables involved mesh together. Only when this is accomplished will a sound design result.

Effect of Method of Construction

103. Dike embankments, classified according to general construction methods, are listed in Table 11. The choice of construction will be governed by available materials, foundation conditions, and economics. As can be seen in Table 11, there are basically three types of embankments with respect to material placement and compaction: compacted, semicompacted, and uncompacted. Classification by these means does not necessarily refer to the end quality of the embankment, rather it specifically refers to how much compaction effort and water content control was applied in construction of the embankment. For instance, both a cast dike and a hydraulic fill dike are classified as uncompacted. However, a hydraulic fill sand dike will have a higher density than will a cast dike built of previously dredged material. The classifications given in Table 11 merely provide a convenient means of grouping dikes according to construction methods. Basically, though, the dike section will increase in size as one goes from a compacted to an uncompacted dike. One exception to this is a low cast dike that is often built with fairly steep side slopes. From a stability point of view, however, these are the least desirable types of dikes. Methods of construction are discussed in more detail in Part VIII.

Basic Design Concepts for Slope Stability

104. There are three basic concepts of dike design for slope stability. These are shown in Figure 14 and are termed floating,

Table 11

Dike Classification According to Method of Construction

Type Compaction	Method of Construction	Requirements, Use, and Remarks

Compacted	Hauled, spread, and compacted with compaction equipment
	Requires specification of:
	  Water content with respect to optimum
	  Loose-lift thickness
	  Type compaction equipment and number of passes
		Requirements:
		a. Strong foundation of low compressibility
		b. Fill materials with natural water content reasonably close to specified ranges
		Provides:
		a. Steep-sloped embankment, occupying minimum space
		b. Strong embankment of low compressibility

Semicompacted	Hauled or cast with draglines Compacted with fewer passes of light roller or controlled traffic of hauling, spreading, or shaping equipment Fill material placed at natural water content (i.e., no water content control) Usually placed in thicker lifts than compacted method	Used where: a. Steep-sloped compacted embankments are not required b. Relatively weak foundations exist that cannot support steep-sloped compacted embankments c. Underseepage requirements are such as to require a wider embankment base than is necessary for compacted embankments d. Water content of fill material or amount of rainfall during construction season is such as to not justify compacted embankments, but low enough to support equipment
Uncompacted	Hauled (dumped in place), cast, or pumped hydraulically Little or no spreading or compaction Usually shaped to final lines and grade No lift thickness control Fill material placed at natural water content (i.e., no water content control)	Used where: a. Nearby materials are inadequate for compacted or semicompacted construction b. It is the most economical method of placement c. Dike heights are low for cast or dumped-in-place methods d. Relatively weak foundations exist e. Embankments with wide bases are required for stability (for pumped methods)
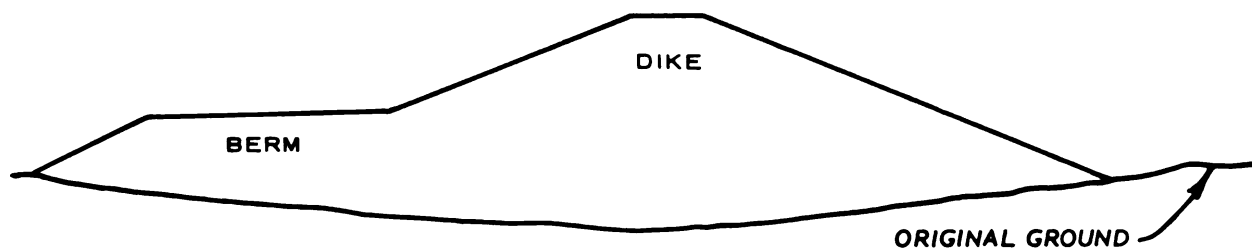
a. FLOATING SECTION
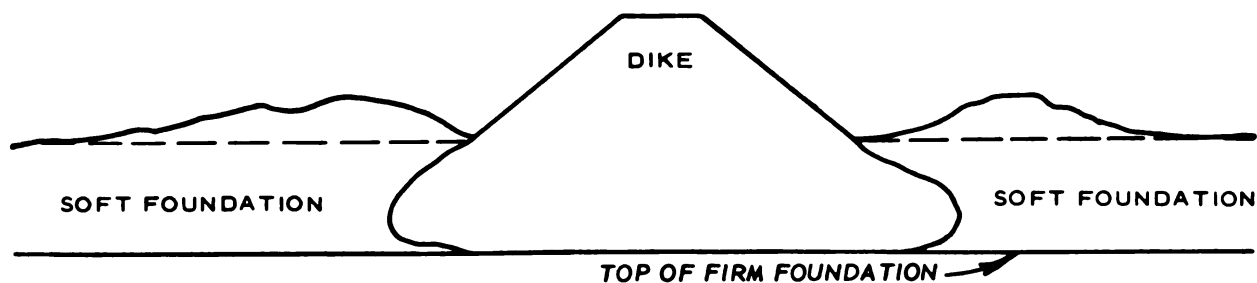
b. DISPLACED SECTION
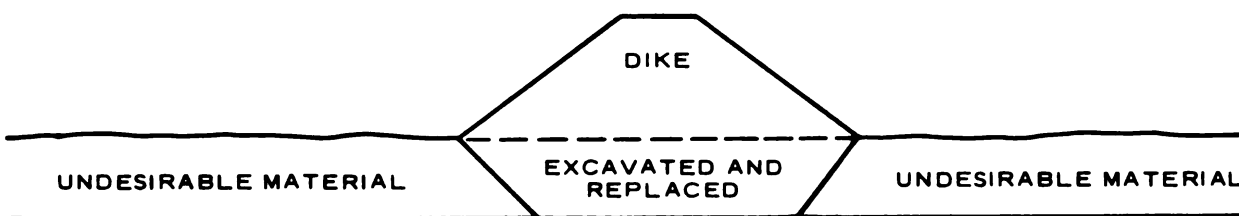
c. SECTION FORMED BY EXCAVATION AND REPLACEMENT

Figure 14. Basic methods of forming dike sections for stability

displacement, and excavation and replacement. There are many variations
of these basic concepts, especially of the section built by floating,
which can be used on any type of foundation. The displacement and the
excavation and replacement sections are applicable, respectively, to
very soft foundations and to foundations containing soft, organic, or
otherwise undesirable material to a reasonably shallow depth. These
basic concepts along with combinations and variations are discussed in

detail in Parts VII and VIII. The determination of which method to use is based on available embankment materials and foundation conditions.

Floating method

105. The floating section gets its name from use on soft founda-tions but is applicable to stronger foundations as well. The concept involved with this type of section is to spread the embankment load sufficiently by the use of flat slopes and berms so that the foundation is not overstressed. This is usually an economical method of design but becomes more uneconomical as foundations become weaker, due to the increase in material required. Geometry of the section is determined primarily by stability analyses.

Displacement method

106. Dike construction by the displacement method is just the opposite of the floating technique in that it purposely overstresses the soft foundation material until it fails and is displaced by stronger fill material. This method requires the existence of very soft founda-tion materials (undrained strengths less than about 150 psf) that will readily fail and displace. It is desirable to have a stronger material underlying the soft material, but the method can be used in deep nor-mally consolidated materials.

Excavation and replacement method

107. Specifying a dike section to be constructed by excavation and replacement techniques is a positive means of ensuring stability. This method involves excavating soft or undesirable material and re-placing it with more desirable material. It is, however, limited by the depth of undesirable material and location of the water table, as it becomes more uneconomical as the thickness of material to be removed and replaced increases and, if dewatering is required, the higher the groundwater table. Generally, 20 ft is about the limit of excavation in the use of this technique. This method requires the existence of a firm base (stronger material) under the undesirable material.

Raising of Existing Dikes

108. Due to the weakness of many dike foundations, the height to

depends largely on imposed loading conditions, i.e. the same foundation may be perfectly stable under one loading but inadequate under another. However, foundation deposits that are prone to cause problems may be broadly classified as follows: (a) very soft clay, (b) sensitive clay, (c) loose sand, (d) natural organic deposits, and (e) man-made organic deposits.

146. Very soft clay is susceptible to shear faulure and excessive settlement. Sensitive clay is brittle and, even though possessing considerable strength in the undisturbed state, is subject to partial or complete loss of strength upon disturbance. Fortunately, extremely sensitive clay is rare in the United States. Loose sand is also sensitive to disturbance and may liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of slopes. Most organic soils are very compressible and exhibit low shear strength. The physical characteristics of natural organic deposits such as peat can sometimes be predicted with some degree of accuracy. Highly fibrous organic soils with water contents of 500 percent or more generally consolidate and gain strength rapidly. The behavior of organic debris deposited by man, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict.

147. The following paragraphs discuss methods of dealing with foundations that are inadequate from the standpoint of available shear strength for construction of proposed dikes. These methods are excavating and replacing poor materials, displacing undesirable material by end-dumping fill material, constructing the dike in stages to permit consolidation of the foundation, densifying loose sand, flattening embankment slopes, and constructing stability berms.

148. <u>Excavation and replacement.</u> The most positive method of dealing with excessively weak and/or compressible foundation soils is to remove them and backfill the excavation with more suitable material. This procedure is usually feasible only where deposits of unsuitable material are not excessively deep (i.e. up to about 20 ft in thickness), where suitable backfill material is available, and where a firm base exists upon which to found the backfill. The excavation and

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replacement can be accomplished by any practical means, but for most dikes in areas of high water tables (i.e. marshes, tidal flats, etc.) excavation is best accomplished with dredges, matted draglines, and barge-mounted draglines. Where backfilling is to be accomplished in the wet, only coarse-grained material should be considered for use as back-fill. The amount of excavation need not always be under the entire section or to full depth of soft material, but can be partial if determined by stability analyses to be appropriate. Some sections successfully used in the past to prevent horizontal sliding of the embankment are shown in Figure 37. Excavation and replacement should be considered wherever possible.
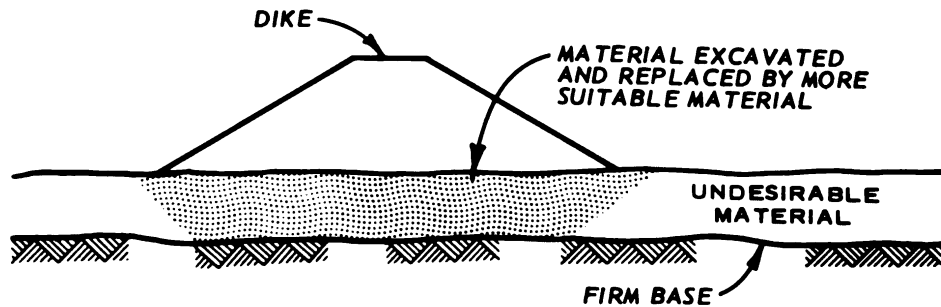
a. COMPLETE EXCAVATION AND REPLACEMENT
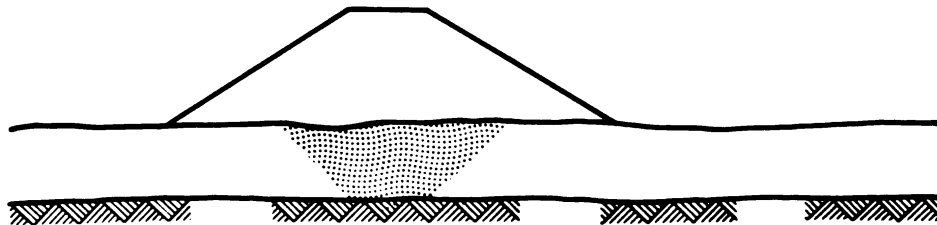
b. PARTIAL EXCAVATION AND REPLACEMENT

Figure 37. Typical use of excavation and replacement
method to improve stability

149. <u>Displacement of undesirable material by end-dumping fill.</u>
Dikes must frequently be built over areas consisting of very soft materials. Although the depths of these deposits may not be great, the cost of their removal may not be justified and a dike having adequate stability can be constructed by end-dumping fill and utilizing it's weight to displace the undesirable material.

150. It is desirable to use this method where a firm bottom exists at a reasonably shallow depth; it has, however, been successfully employed in areas where no definite firm bottom existed, but the displaced material merely increased in strength with depth, in which case the depth of displacement is considered to be that necessary to stabilize the embankment at the desired height (Figure 38). However, use of the

a. WITH FIRM BOTTOM

*STRENGTH INCREASES WITH DEPTH

b. WITHOUT FIRM BOTTOM
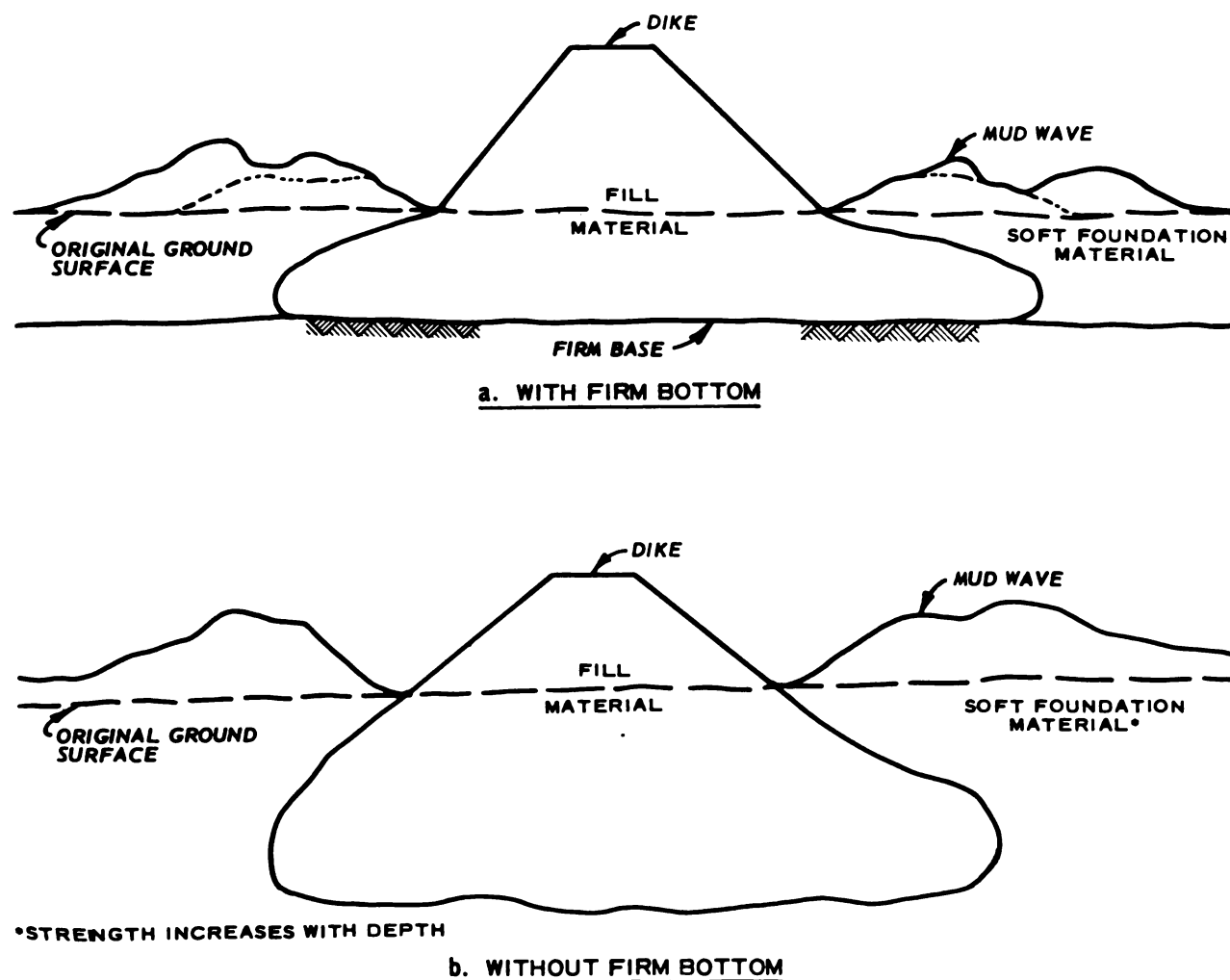
Figure 38. Final dike sections after displacement of soft foundation material

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displacement method in the latter case does increase the likelihood of post-construction settlement.

151. Due to the construction techniques required to successfully use this method, it is highly desirable to place fill by end-dumping methods rather than by hydraulic means. It is also desirable that the material to be displaced exhibit some sensitivity and have average in situ shear strength of less than about 150 to 200 psf. The greater the sensitivity of the material and the lower its in situ strength, the easier it is to displace.

152. Basically, the displacement technique consists of advancing the fill along the desired alignment by end-dumping and pushing fill over onto the soft material with dozers, thus continually building up the fill until its weight displaces the foundation soils to the sides and in front of the fill (Figure 39). By continuing this operation, the dike can finally be brought to grade. Since this method involves the encouragement of foundation displacement, the section should be as steep sloped as possible and built as high as possible as it advances across the foundation. The fill should be advanced with a V-shaped leading edge so that the center of the fill is always the most advanced, thereby displacing the soft material to both sides (Figure 40). This will greatly lessen the chances of trapping soft material beneath the fill. A wave of displaced material will develop (usually visible as is evidenced by the photograph in Figure 41) along the sides of the fill. These mud waves have been known to be as high as the top of fill; however, they should not be removed.

153. A disadvantage of this method is that all the soft material may not be displaced, which could result in slides as the embankment is raised and/or differential settlement after construction. Another disadvantage is that final in-place quantities are difficult to determine due to an appreciable amount of fill material being below the ground surface. It is therefore recommended that quantities be based on excavated yardage or provisions be made to take borings after construction or, where the displacement is not too great, settlement plates be installed beneath the proposed alignment prior to construction. All of

a. Coarse-grained fill

b. Fine-grained fill

Figure 39. Shoving fill onto soft foundation
with dozers

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a. PLAN

b. PROFILE
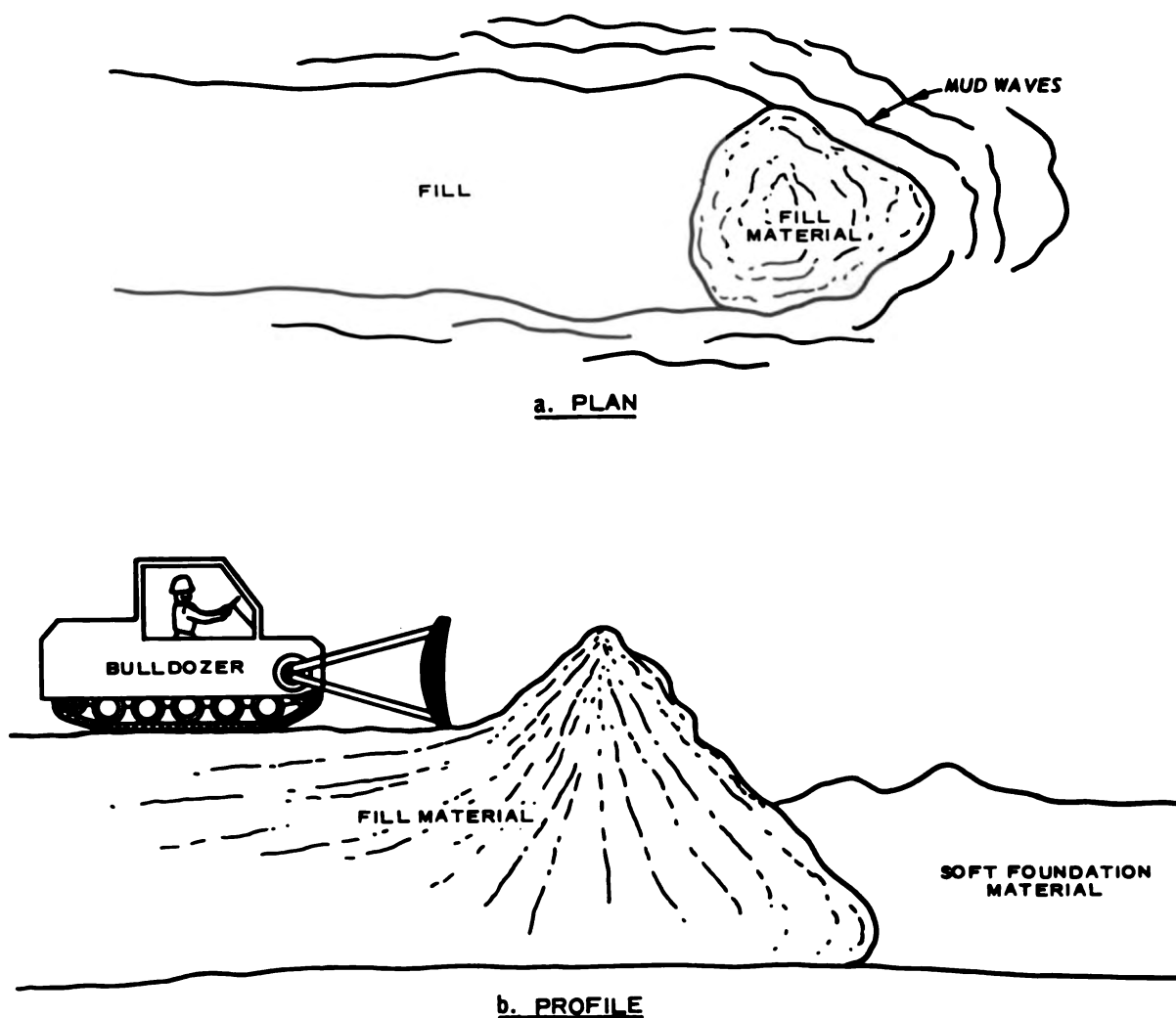
Figure 40.   Advancement of fill using end-dumping and
displacement technique

the above techniques for determining pay quantities have been success-
fully employed for displacement construction in the past.

154.   If a surface root mat or a desiccated layer exists immedi-
ately over the soft material to be displaced, it should be broken up
prior to fill placement. Since this type of construction produces
essentially uncompacted fill, the design of the dike section must take
this into account.

155.   When this method of foundation treatment is being considered
for long reaches of dikes over deep deposits of soft sensitive clays,
the possibility of facilitating displacement by blasting methods should
be evaluated (see Blasters Handbook[21] for general information on

a. View parallel to dike
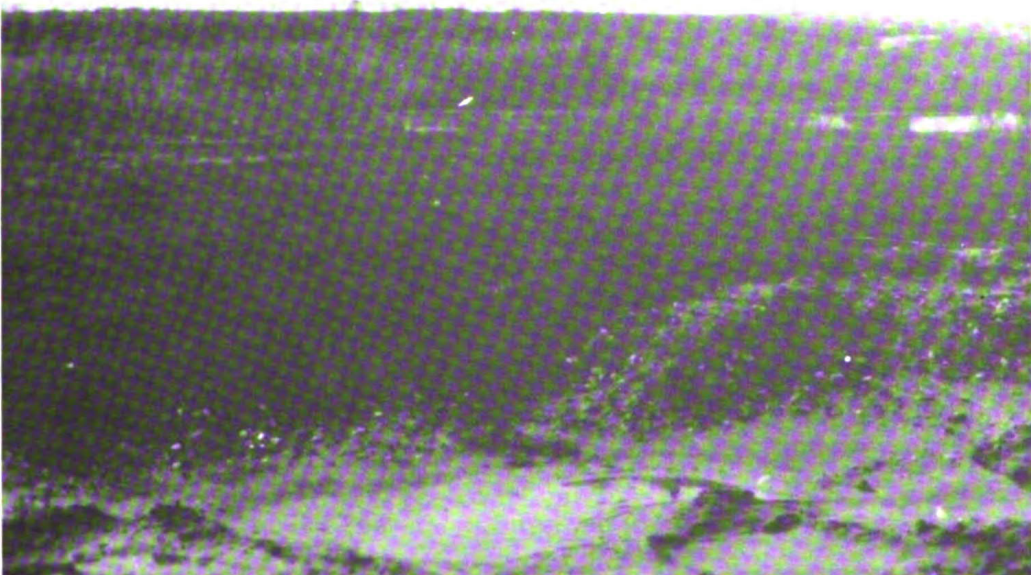
b. View perpendicular to dike

Figure 41. Mud waves from displaced material

blasting used to displace soft materials). Generally, the greater the required depth of displacement, the more economical the blasting method becomes.

156. <u>Stage construction.</u> Stage construction refers to the building of an embankment in increments or stages of time. This method of construction is used when the strength of the foundation material is inadequate to support the entire dike if built at one time. Using stage construction, the dike is built to intermediate grades and allowed to rest for a time before placing more fill. Such rest periods permit dissipation of pore water pressures and consolidation that results in a gain in strength so that higher dikes can be supported. Obviously, this method is most appropriate for foundations that consolidate rather rapidly. This procedure works best for clay deposits interspersed with continuous seams of highly pervious silt or sand. However, lack of speed of consolidation may not be a drawback if the filling rate of the disposal area is slow enough to allow considerable time between construction of the various dike stages. In fact, stage construction appears to be a promising method of constructing retaining dikes as the intervals of construction can, in many cases, coincide with the filling of the disposal area; i.e., full dike height may not be needed until many years after initial construction.

157. In using stage construction, estimates of strength gain with time should be made as described in paragraph 143b(2). Also, it is highly desirable to have piezometers available to monitor the dissipation of pore water pressures. Disadvantages of this method include the need for separate construction contracts and uncertainties with respect to the gain in strength with time.

158. <u>Densification of loose sand.</u> In seismically active areas, the possibility of liquefaction of loose sand deposits in dike foundations may have to be considered. Since methods for densifying sands such as vibroflotation, blasting, etc., are costly, they are generally not considered except for dikes where the consequences of failure are very severe or at locations of important structures in the diking system. However, less costly defensive design features may be provided,

Smith (1959)

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Twelfth and N Streets
Sacramento*

CONTENTS

FROM THE CHIEF JUSTICE

SUPREME COURT OF THE UNITED STATES
Washington 13, D. C.

CHAMBERS OF
THE CHIEF JUSTICE

November 14, 1955

MR. K. C. ADAMS, *Editor*
California Highways and
Public Works

Dear Ken: Enclosed you will find the card requesting the *California Highways and Public Works.* I would feel lost without it. For me, it is as exhilarating as a western thriller. I read and reread every issue because it reflects more than any other publication the growth, development and progress of California.

With best wishes to you and all of your associates in the Department of Public Works, I am

Sincerely,

EARL WARREN

"Open Water" Fill

Unique Project Is Nearing Completion

By VINCENT O. SMITH, Senior Highway Engineer

On November 9, 1955, a contract was awarded to Guy F. Atkinson Co. for completion of the grading of one of the most unusual and interesting highway projects ever attempted. This portion of Bayshore Freeway, between the intersection of Third Street and Bayshore Boulevard in San Francisco and South San Francisco, will cross an arm of San Francisco Bay approximately two miles wide, bypassing one of the most congested sections of highway in the Bay area.

The need for additional highway facilities to handle the increasing traffic between San Francisco and the fast developing peninsula area became apparent in the mid-1930's and numerous traffic studies were made to determine the type and extent of expansion that would best alleviate the growing congestion. Due to the highly developed industrial sections, substandard alignment, grades, and con-

stricted right of way on the existing route through the Visitacion Valley area, it was determined that the most economical and desirable solution was to bypass this area with a new location. This would provide two facilities through this area with a new freeway for through traffic and the existing route to serve local traffic.

Several Routes Studied

Studies of several routes bypassing this area led to recommendation of the present route. It subsequently was adopted and declared a freeway by the Highway Commission in July, 1941.

Since the new route crossed an arm of San Francisco Bay, with underlying mud ranging in depth from a few feet to nearly 80 feet, construction presented a major problem. Comprehensive studies were made to determine the most feasible and economic type

of method of building this project. After eliminating the possibility of a causeway the two methods most carefully analyzed and compared were: (1) displacing the mud with dry fill by end dumping and (2) several variations of predredging the mud to provide a reasonably stable embankment with a minimum of mud displacement.

Because of large cuts on each end of the project and the fact that an ample quantity of borrow material was readily available from nearby sources, it was determined that substantial savings would be realized if the end dump mud displacement method would provide a stable embankment. Since this method of construction had never been used on such a large scale with dimensions and conditions resembling those to be encountered, it was questioned whether the fill could be successfully con-

SAN FRANCISCO BAY
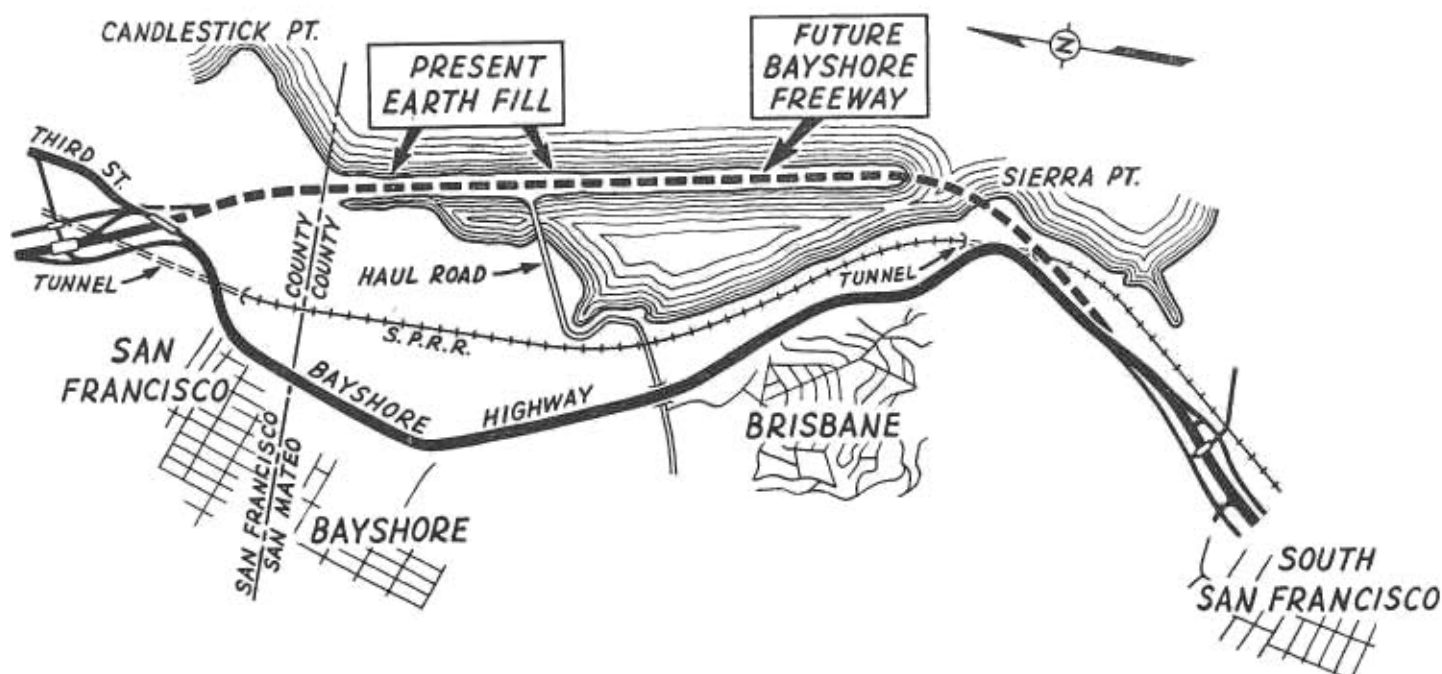
Fill for open water section of Bayshore Freeway across Candlestick Cove. Widening at center of picture is location where reinforced concrete box culverts will be constructed to equalize water level.

structed in this manner. Hence to determine the feasibility of the proposal funds were made available and a contract was let in January, 1952, to construct an experimental section of fill by end dumping.

Mud Fill

Material for this contract was obtained from the right of way and was placed using 20 cubic yard carryalls and tournapulls. The fill was advanced on a 400-foot-wide front in an attempt to float the fill with a minimum of mud displacement. As the fill progressed, it was determined by borings that much greater penetration and displacement of mud was occurring than had been originally estimated. Calculations showed this greater penetration would allow the width to be reduced and still obtain reasonable stability so

the fill was advanced further into the bay at a width of 300 feet.

Reducing the width caused greater displacement, so the fill was narrowed again. The remainder of the experimental fill unit was constructed 250 feet wide, being completed in August, 1952.

Based on the success of obtaining a reasonably stable fill over mud of a maximum depth of 40 feet on this first contract, a second experimental project was recommended to be placed by the same method to determine the feasibility of construction over mud which reached a depth of 80 feet.

Overhead Crossing

This second contract was awarded in June, 1953, and it included building an overhead crossing over eight tracks of the Southern Pacific Railroad and

nearly two miles of haul road to a borrow site west of the existing Bayshore Highway. To reach the center line of the proposed freeway fill it was necessary to cross 1,200 feet of the bay with the haul road which was to be constructed 30 feet wide over mud that reached a depth of 60 feet. Construction of a fill of this width resulted in nearly 100 percent displacement of the soft bay mud and provided a road over which nearly 3,000,-000 cubic yards of fill material has been hauled with only normal grading for maintenance.

The successful completion of this haul road confirmed further the feasibility of the method of construction, so instead of feasibility, our main concern during construction of the second experimental fill became placing the fill in such a manner as to obtain a uniform displacement of mud both laterally and longitudinally.

Uniform Displacement

If the fill could be placed so that a uniform displacement of mud could be obtained, differential settlement would be a minimum and only normal maintenance would be required.

Borings were made during construction to determine the depth of displacement, and records of quantities and methods of placement were correlated with these borings to determine factors affecting displacement.

Numerous variable factors were found that influenced displacement, the prime ones being:

1. The shape of the advancing face of the embankment.
2. The type of equipment used to place the fill material.
3. The rate at which the fill was placed.
4. The elevation at which the fill was carried.
5. Stoppages.
6. The type of material of which the fill was constructed.
7. Strength of the underlying mud.
8. Depth of the underlying mud.
9. Tide action.

A change in any of these factors caused others to vary and resulted in a change in displacement. Controls had to be established and varied during construction to meet the conditions at hand.

McFarlane (1969) excerpts

Muskeg Engineering Handbook

By the Muskeg Subcommittee
of the NRC
Associate Committee on
Geotechnical Research

EDITED BY
Ivan C. MacFarlane

UNIVERSITY OF TORONTO PRESS

over the muskeg, and helped to prevent fill material from penetrating the surface of the muskeg. Wire mesh, dried bundles of peat, and bundles of straw have also been used as a base for fill.

As the frequency and weight of traffic became greater, the road standards increased. Since the composition of peat and the depth of the deposits were variable, differential settlement under this fill was frequently a serious problem.

In the nineteen thirties, the Michigan State Highway Department pioneered the method of excavating and displacing the peat (Cushing and Stokstad 1934). In the shallower deposits, excavation was frequently performed with draglines or shovels and the excavated area backfilled with inorganic soil, preferably sand and gravel. In very deep deposits, partial excavation and backfill were frequently used. This technique did not prove too successful because of major settlement and it led to the development of displacement methods of construction. With gravity displacement, the leading edge of the fill was built up to such a point that overloading occurred. The fill material settled, displacing the underlying peat. To assist this process, water jetting was occasionally used to reduce the shear strength of the peat and facilitate displacement. Blasting was also used to displace the peat. Two methods, the underfill and toe shooting techniques, were developed (Parsons 1939).

In the mid fifties, efforts were directed towards construction without removal of the peat. Brawner (1958) reports the results of an experimental road section to test preconsolidation of peat. With this method, a load in excess of that finally proposed is placed to induce settlement. When the rate of settlement decreases significantly, the excess weight is removed. Subsequent settlement is usually small. This procedure has been used successfully on more than ten major projects in Canada. In recent years, several modifications of this method have been developed. Lea and Brawner (1963) report the use of sawdust as lightweight fill to reduce the ultimate load on the peat and to act as a weightless spacer where excessive settlements were expected in the peat. Flaate and Rygg (1964) in Norway outlined a similar approach but used timber as a base for the sawdust.

Where muskeg is deep for short distances or where heavy loads are required, bridges on piles have been used. A summary of the methods of road construction over organic terrain that have been used to date is shown in Table 6.1 (cf. Mac-Farlane 1956).

6.2   PRELIMINARY DESIGN CONSIDERATIONS

The method of construction selected should be the one that provides the desired standard of road to serve the user efficiently at the lowest possible cost. Numerous methods are available, each of which has explicit applications and limitations.

The first factor that should be evaluated is the standard of highway to be constructed. If only local traffic is to be served, considerable settlement, undulation, distortion, etc. can be tolerated. Under these conditions, construction of the road

Prior to construction, all woody vegetation should be cut off at ground level, with the smaller cuttings placed as a mat on the surface. Under no circumstances should the surface mat of muskeg be disturbed. Trees or stumps should not be burned on the muskeg because the fires may spread and continue for many months or years. Figure 6.5 shows a typical cleared portion of the right-of-way through muskeg.

If the muskeg is very weak and has little or no surface mat, it may be necessary to place corduroy or use fascine construction. Logs placed side by side or criss-cross are most common. Other materials used are planks, brush, wire mesh, dried peat, bundles of straw, sawdust, or wood chips. Their purpose is to spread the load of the fill, provide some buoyancy, and prevent the fill from sinking into the muskeg.

Granular soils are preferred for fill on top of muskeg. They are easy to place, grade, and compact and they provide better subgrade support. Rock has a tendency

FIGURE 6.5   Clearing of the right-of-way completed prior to the use of preconsolidation construction on the Trans-Canada Highway near Chilliwack, BC

FIGURE 6.6   Gravity displacement of organic lake-bed soil near Revelstoke, BC

A modification to complete excavation which has been used in peat over about 10 feet (3.05 m) in depth is to excavate a portion of the peat and backfill with stable material. This is often described as "partial excavation." The general procedures for design and construction combine those used for floating the road and for complete excavation. The amount of settlement that will occur, however, can be large. The weight of the fill placed is usually considerably in excess of the weight of the peat excavated and this increase in weight contributes to the increase in settlement. The weight can be reduced by using some lightweight fill. If the physical characteristics of the peat differ and the depth of the deposit changes over relatively short distances, moderate to severe differential settlement which will continue for several years can be expected. With this method, it is desirable that the construction of the pavement surface be delayed as long as possible to allow most of the settlement to take place.

(3) Displacement Methods
A Displacement by Gravity

Gravity displacement of peat has been carried out on many projects, particularly in the United States. As the fill is constructed across the muskeg, its frontal face is advanced with a "V" shape and surcharged sufficiently so that a shear failure takes place, displacing the peat laterally. A typical example near Revelstoke, BC, is shown in Figure 6.6. This technique is most successful where the peat is 10–20 feet (3.05–6.1 m) in depth and has a low shear strength. If the peat is relatively stiff and difficult to displace, additional weight may be obtained by continuous watering of the fill.

A more frequent technique for reducing the shear strength is jetting. Early attempts at jetting required that the fill be placed first and that jets be forced down through the fill into the peat below. The additional weight of the fill and consolidation of the peat made it difficult to bring the peat to a fluid consistency. Greater success is most often achieved with water pumped into the peat before the fill is placed, during placement, and as long thereafter as the fill continues to settle. Occasional borings may be necessary to indicate when extra jetting is necessary. The spacing of the jets depends on the type and depth of peat. Experience in Michigan has indicated that for best results jets should be spaced 10–25 feet (3.05–7.62 m) apart over the entire area of the proposed fill (Cushing and Stokstad 1935). Clean sand and gravel fill is required. Clayey soils will tend to become fluid and to displace laterally. Centrifugal pumps with a capacity of 500 gallons (2272 litres) of water per minute operating at a 250-foot (76.3-m) head and 1450 r.p.m. can operate 12 to 15 jets per minute.

Care is required with jetting and gravity displacement techniques since pockets of peat, which will lead to future settlement may be left. As a result of the increased water content of the peat which is displaced laterally, the weight of the fill may cause considerable lateral compression, with the result that the shoulders may settle and move horizontally at a gradually decreasing rate for many years.

FIGURE 6.26   Displacement fill area at Pontiac with pond formed by consolidation and displacement of peat. Uplifted area is at right

In this case the excessive displacement was caused not only by the slope of the underlying firm surface but by the presence of the soft silt layer. Differences in stress–strain characteristics of peat and soft silt or clay result in the cohesive soils failing before the full strength of the peat is mobilized. Hence, the effective strength of the peat is actually decreased by the presence of weak cohesive soils and a careful engineering analysis of stability is required. More experience with displacement fills on sloping bases is needed before fill quantities can be predicted, even approximately.

The displacement methods reviewed below are also discussed in Section 6.5.

A   Displacement by Gravity

Gravity displacement is the basic method. It is best done with a rolling surcharge at the front end of the fill, advanced with a narrow end, as shown in Figure 6.27. The technique is most successful where the peat is 10–20 feet (3–6 m) deep with low strength.

The effectiveness of gravity displacement can be increased by various techniques. The simplest of these (Casagrande 1966) uses the method shown in Figure 6.28. The surface crust of muskeg is broken up over a width of 5–10 feet (1.5–3 m) along the centre line, preferably by blasting. A narrow fill is then end-dumped along this line followed by the fill over its full width, thus ensuring that a narrow core of fill penetrates deeply into the muskeg. This method can also be used at the start of a fill operation that uses displacement by blasting.

Other techniques to improve displacement are intended either to increase the weight of the fill or decrease the strength of the peat. The weight of a granular fill can be increased in place by jetting water into it until it is nearly saturated. This procedure has been used in combination with a general surcharge to facilitate displacement. The surcharge is later bulldozed to the side.

Methods to decrease the strength of peat are disturbance with water jetting or blasting. Both of these methods help the peat to move out from under the fill more rapidly and completely.

B   Displacement by Jetting

This method is used to break up and soften the peat preferably both before and after the placement of fill. Although effective, it requires suitable pumps, piping and jets, and continuous careful supervision. Sufficient water is usually available.

C   Displacement by Blasting

This method uses explosives to weaken and displace the peat and is much better adapted than jetting to regular construction procedures. Two methods are in use and are described in Section 6.5 and elsewhere (Canadian Industries Limited 1964).

   The underfill method is used to muskeg depths of 30 feet (9 m) and more. It may be carried out by putting explosive charges down from the toe of the fill to the required position and then pushing the fill forward before detonation (Casagrande 1966). If further incentive is needed to induce the peat to flow to the sides of the fill, side ditches may be blasted or dug to relieve the lateral pressure.

   The toe shooting method (Section 6.5(3)) is used up to depths of 20 feet (6 m) in soft peat and deeper when the technique of torpedo blasting is employed.
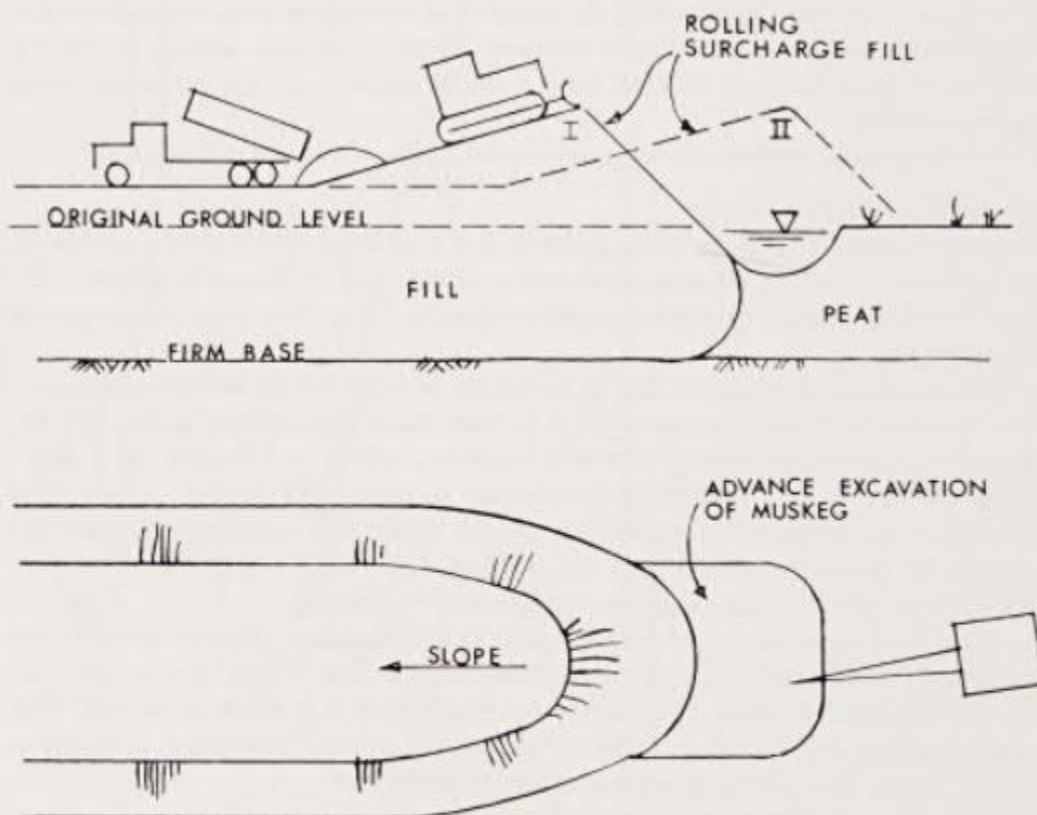

FIGURE 6.27   Gravity displacement method of fill using rolling surcharge and relief excavation at front

Holtz (1989) excerpts

TREATMENT OF PROBLEM FOUNDATIONS FOR HIGHWAY EMBANKMENTS

ROBERT D. HOLTZ
Professor of Civil Engineering
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Topic Panel

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TERRY L. YARGER, *Montana Department of Highways*

treated soft material may cause significant nonuniform postconstruction settlements, and thus partial excavation may not be desirable for high-quality roads except at sites where the compressible layer is relatively homogeneous and approximately the same thickness, and the settlements that do occur are tolerable. Such sites obviously require a thorough site investigation and testing program.

In addition to settlement problems, the partial excavation method frequently involves stability problems. The underlying unexcavated soft materials may be too weak to support the embankment without stage construction, berms, lightweight fill, or very flat slopes. Furthermore, the excavation process itself may leave accumulations of soft material on the bottom or it may disturb the underlying soft soil. Embankment stability analyses (Chapter 6) with failure surfaces extending to the unexcavated soft material should be performed.

Underwater Fill Placement

Clean sand or sand and gravel with less than 8 to 12 percent finer than a $75\mu m$ (No. 200) sieve is well suited for underwater placement because these materials are not sensitive to placement water content. Materials containing more fines can be a problem because of segregation and density control, but they also can be upgraded, as described by Sinacori et al. (33). Densification of backfill materials underwater can be accomplished, and Johnson et al. (55), Arman (36), and Broms (31) describe some possibilities, although none are very cheap. Recent developments are reviewed by Koning (56) and Dembicki (57). One of the strong advantages of using blasted rock for backfill underwater is that for modest thicknesses, it is not very compressible, even when loosely dumped. To protect the underlying clays during dumping operations, a layer of sand and gravel or a geotextile could be used as a separator layer. If a geotextile is used, it would have to have a very high "survivability" (58) to avoid being damaged during backfilling.

A variety of methods may be used for placing fill materials underwater, and they are listed in Table 9. Construction inspection and control procedures suitable for excavation and fill placement were given in Table 8. Effective construction control for excavation or dredging and fill placement has an identifiable economic value because it affects directly the amount of material excavated and hence the volume of backfill that must be placed. Differences in quality control as they relate to the volume of required excavation and hence volume of fill are discussed for highway construction by Johnson et al. (55). The quality of construction control provided affects the overall cost of the job far more than do the differences in cost of improved inspection. In addition, poor construction control may result in a rough-riding pavement because of postconstruction settlements from the consolidation of soft entrapped or unexcavated materials that would have been avoided by better construction supervision.

Removal by Displacement and Partial Excavation

As an alternative to excavation, it may be possible to displace soft materials by deliberately overstressing and therefore failing them by the weight of embankment combined with a temporary surcharge and/or partial excavation. This method is illustrated

TABLE 9

UNDERWATER FILL PLACEMENT METHODS (55)

METHOD	CHARACTERISTICS
Bottom-dump scows	Fill assumes flat slopes unless retained.
	Limited to minimum depths of about 15 ft (4.5 m) because of scow and tug drafts.
	Rapid; quick discharge entraps air and minimizes segregation.
Deck scows	Usable in shallow water.
	Unloading is slow, by dozer, clamshell, hydraulic jets, conveyors.
	Steep side slopes of fill can be achieved.
Hydraulic	Coarse materials drop out first; may cause shear failures in soft foundations.
	Fines may collect in low areas and have to be removed.
	Inspection of material being placed may be difficult.
Dumping at land edge of fill and pushing material into water by bulldozer	Fines in material placed below water tend to advance and accumulate in front of advancing fill.
	Work arrangement should result in central portion being ahead of side portions to displace sideways any soft bottom materials.
	In shallow water, bulldozer blades can shove materials downward to assist displacement of soft materials that accumulate at toe of fill. (Not suitable for displacing unexcavated soft materials.)

in Figures 6 and 7, and it is described by Sinacori et al. (33), Moore (34), Arman (36), Broms (31), and Hartlén (41). Weber (59) describes a reasonably successful project in which 60 ft (18 m) of soft San Francisco Bay mud were stabilized by displacement.

Removal by displacement is an old technique; it was not uncommon for fill to be continually dumped on a soft marshy area until the roadway eventually stabilized. Usually a large mud wave was created to the sides and ahead of the fill. As noted in Synthesis 8 (4), the method is not used so often today because of the uncertainty of complete removal of the undesirable soft materials. The questionnaire indicated that almost half the states have used displacement methods for both stability and settlement problems.

Broms (31) shows some procedures used in Sweden to control the direction of the displaced soil mud wave (Figure 8). Because the displacement method relies on the progressive failure of the foundation soils, the work must be carried out continuously. Thus it is important during even short breaks in the operation that personnel and equipment be kept some distance back of the leading edge. Any upheaved marsh material that accumulates at the leading edge of the fill should be removed to avoid

FIGURE 6   Marsh displacement (45 ft or 14 m deep) and embankment construction to surcharge grade in Michigan (3).

entrapping pockets of the displaced soil within the embankment. Although in some cases excellent removal of the soft soil down to firm bottom may be achieved, elsewhere pockets of soft soil may remain, which results in differential settlements and poor embankment performance. This is similar to the problem of incomplete excavation discussed above, and techniques discussed in that section apply here also.

Another problem is that the displacement of soft subsoils by the weight of the embankment may result in the intrusion of fill into the soft soils outside the limits of the roadway, which would add to the cost of the construction. However, there is good evidence that this does not occur to any appreciable extent if the mudwave is properly controlled (34, 60). The mudwave and possible surface organic mat should be removed from in
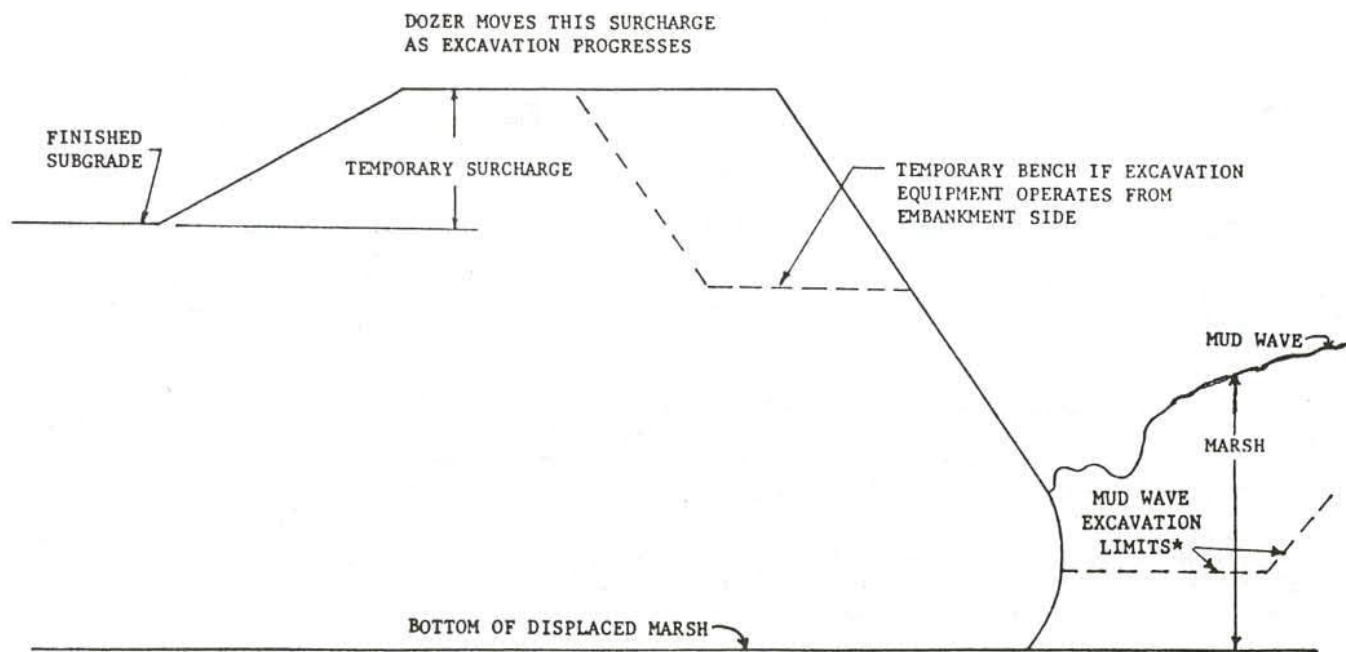
FIGURE 7   Longitudinal section of marsh removal by displacement and embankment construction with surcharge (3).

(a) Symmetrical nose

(b) Unsymmetrical nose

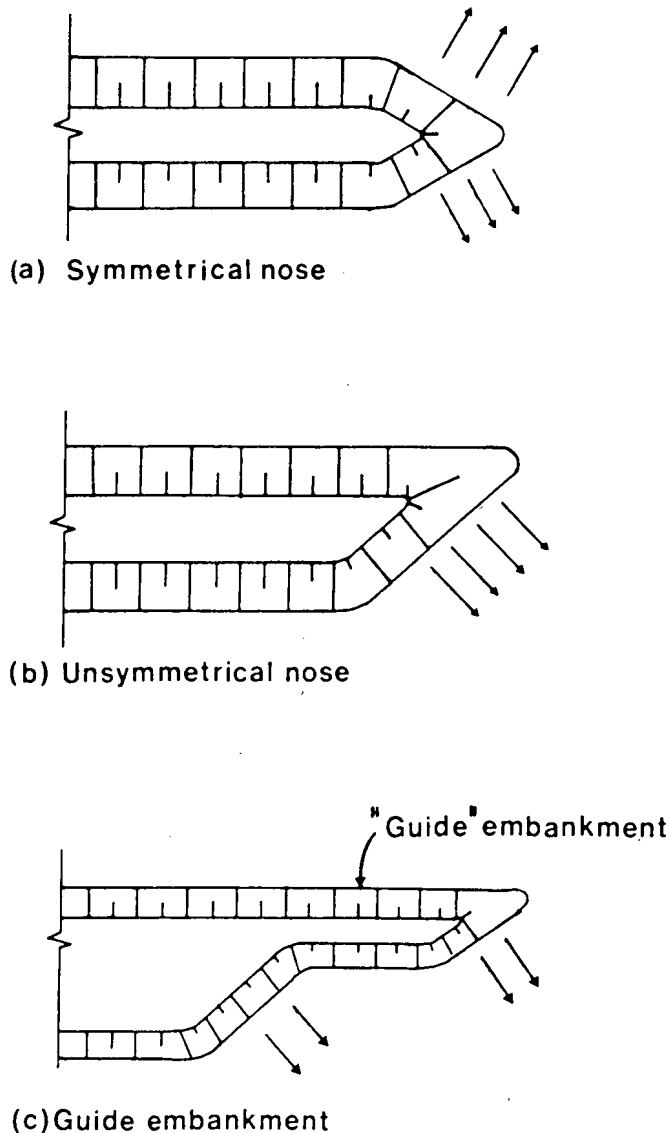"Guide" embankment

(c) Guide embankment

FIGURE 8   Directing the displacement of soft soils (31).

front of the fill for a minimum distance of 50 ft (15 m); then all displacements will go in this direction and there will be very little sideways displacement. Moore (60) recommends excavating any of the mudwave appearing above the water surface.

Recent research at the Swedish Geotechnical Institute (41) has shown that (a) a large embankment height results in a more pronounced failure of the subsoil, (b) because of smaller subsoil displacements, a narrow embankment penetrates more easily, (c) in very wide embankments, the displacement is primarily ahead of the leading edge, rather than to the sides. This results in a need for excavation and blasting (next section). The Swedish research also found that the method is *not* recommended when (a) the overall stability of the area is low, (b) the firm bottom is steeply inclined ($\geq 45°$) perpendicular to the centerline (a slide could occur sideways), (c) the strength increases substantially with depth (this prevents penetration and displacement and results in long-term settlements), and (d) clays of high strength and high sensitivity (quick) are present that are subject to sudden, rapid sliding.

The displacement method should be designed just like any other alternative; recommended procedures are given in Chapter 6. Its success may depend on the sensitivity of the soft soils to remolding, and perhaps that is why the method has been used with such success in Sweden. Just as with any other treatment technique, selection of this method should follow a detailed evaluation of it and other alternatives. Available construction control and inspection and the consequences of delays and of postconstruction settlements are very important considerations.

Displacement of Soft Materials by Blasting

The displacement of soft materials by blasting has been attempted on numerous occasions, and as mentioned above, it has been used in connection with removal by displacement. Thus, its use must be augmented by controlled placement of foundation and embankment fill. The technique is described by Sinacori et al. (33), Casagrande (61), Arman (36), Broms (31), and Hartlén (41); the paper by Casagrande provides the most extensive discussion of procedures for displacement blasting.

The blasting technique requires expert and constant field supervision to assure that adjustments to blasting procedures are made as conditions warrant. Unless this is done, soft material may not be completely and properly removed. This method is considered to be sufficiently sensitive and difficult to use so that it should *not* normally be considered as an appropriate alternative. Only three states report having used blasting, with one (Wisconsin) remarking, "not in the last 25 years!"

Embankment Widening

Today there is much more emphasis on upgrading of existing facilities rather than construction in completely new locations. Some of the problems associated with embankment widening will be described in Chapter 4. All four methods just described might be suitable under the right circumstances for widening existing embankments, provided the construction procedures are feasible. Moore (34) describes one such solution to the problem of widening embankments crossing swamps and marshes (Figure 9). Note that in order to avoid instability of the existing embankment, the length of the open excavation must be carefully controlled by keeping the backfilling operations close to the excavation. Also, the water level in the excavation must be maintained at its original elevation. There are also other soil improvement procedures that would be suitable for stabilizing widened sections to existing embankments, and these will be discussed as appropriate in this synthesis.

FOUNDATION STABILIZATION BY CONSOLIDATION

Components of Settlement

When a soil deposit is loaded by an embankment, both vertical deformations (called settlements) and horizontal deformations will occur in the foundation soil. Because these deformations may adversely affect the performance of the embankment and structures it supports (pavements, bridge abutments, etc.), predictions of these deformations is a primary obligation of the

Calaveras Dam Disposal Site
Displacement Method Construction Photos

March 3, 2013

04/12/2013 13:17

04/12/2013 13:24

Reference line from photo taken 4/12/13

Approx location of scarp on 4/12/13 due to fill slough

Fill advanced from 4/12 to 4/25

Reservoir Ele 678.5 on 4/25/13

Aerial Photo taken 4/25/13

Disposal Site I, 5/9/13

06/01/2013 11:30